# REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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# AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

## PAPERS AND DISCUSSIONS




# THE SEWAGE DISPOSAL WORKS OF DECATUR, ILLINOIS*


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### Synopsis


Since 1917 the Sanitary District of Decatur, Ill., has completed intercepting sewers, pumping stations, and treatment works for sewage disposal at a total cost for construction projects of about \$1 700 000. The treatment of the sewage has been complicated by the waste from a large starch works which has increased the human population of about 50 000 to a total equivalent population of 300 000, more or less, during periods of heavy corn grinding at the starch works. The peculiar character of the sewage has required the operation of two testing stations prior to the design of the sewage treatment works. A record of the steps taken and the works built in connection with the development of this project are described in this paper. The writers' connections with the work were as Consulting Engineer and Superintendent, respectively, for the Decatur Sanitary District.


### General Statement.

*The Problem and the Work Done.*—For a year or so prior to 1912, the pollution of the Sangamon River below Decatur, Ill., was sufficiently marked to arouse complaint and the situation was brought before the Rivers and Lakes Commission, which had jurisdiction over stream pollution in Illinois at that time. The city had a population of about 35 000 and included among its industries a starch works with a daily grind of 5 000 to 10 000 bushels of corn. The sewage from the starch works had a relatively high population equivalent and it was soon recognized that special study would be required to determine the most suitable method of treatment. All the existing sewers were combined, with four main outlets into the river below a low channel dam at the


Note.—Written discussion on this paper will be closed in **February, 1929.**


* Presented at the meeting of the Sanitary Engineering Division, Columbus, Ohio, October 13, 1927.


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water-works (Fig. 1). For several months nearly every year, almost the entire flow of the river was diverted to the water supply and discharged again into the river as sewage. Thus, an increased water supply through storage on the river was indicated.

Projects for intercepting sewers and sewage treatment were developed in 1915 with an estimated cost of $730 000. This relatively high sum made financing difficult, so that ways and means were carefully studied by city officials and citizens' committees. As a result, the Sanitary District Act of 1917* was prepared and passed by the Legislature. All the construction work for sewage disposal has been done under this Act, amounting all told to nearly $1 700 000. The city has since grown to about 50 000 inhabitants and the grind of the starch works to as much as 45 000 bushels in 24 hours.

The work done may be briefly summarized as follows:

(*a*) The necessary field surveys and office studies for the determination of intercepting sewer and sewage treatment projects.

(*b*) The operation of two testing stations. One comprised an Imhoff tank and sprinkling filter to determine the behavior of the mixed domestic and industrial sewage. This station was operated in 1914 and again in 1917. The other comprised a tank with a Simplex surface aerator for studying the partial or pre-aeration of the Imhoff tank effluent before applying it to sprinkling filters at the main sewage treatment plant. This work was done during 1925 and 1926.

(*c*) The construction of an intercepting sewer 18 317 ft. long and a sewage treatment plant comprising a grit chamber, six Imhoff tanks, sludge-drying beds, six pre-aeration tanks, two settling tanks for aerated Imhoff tank effluent, 3 acres of sprinkling filters, one final settling tank, and appurtenances. There are two small pumping stations for low-level districts. Actual construction work was started in 1919 and completed in 1927.

In addition, the District has done a certain amount of cleaning work along the river; has acquired sites and right of way; has investigated coal mine subsidence; has fought numerous legal battles; and has carried on a vast amount of administrative work. Under conditions found at Decatur, the completed sewage treatment plant has a capacity for a population equivalent of 150 000.

*The City and the Sanitary District of Decatur.*—The City of Decatur is on the Sangamon River about 50 miles by river above Springfield, Ill. The drainage area of the river above Decatur is about 862 sq. miles. The city has an area of 4 885 and the District of 21 120 acres. The trend of population is given by the following figures, the design having been based on an ultimate actual population of 120 000:

| Year. | Population.† | Year. | Population.† |
|---|---|---|---|
| 1890 | 16 841 | 1930 | 57 500 |
| 1900 | 20 754 | 1940 | 71 000 |
| 1910 | 31 140 | 1950 | 84 000 |
| 1920 | 43 818 | 1960 | 99 000 |
| 1927 | 53 400 | | |

* *Transactions*, Am. Soc. C. E., Vol. 91 (December, 1927), p. 441.

† The population figures for 1890, 1900, 1910, and 1920, were taken from the United States Census reports; those from 1927 are estimated.

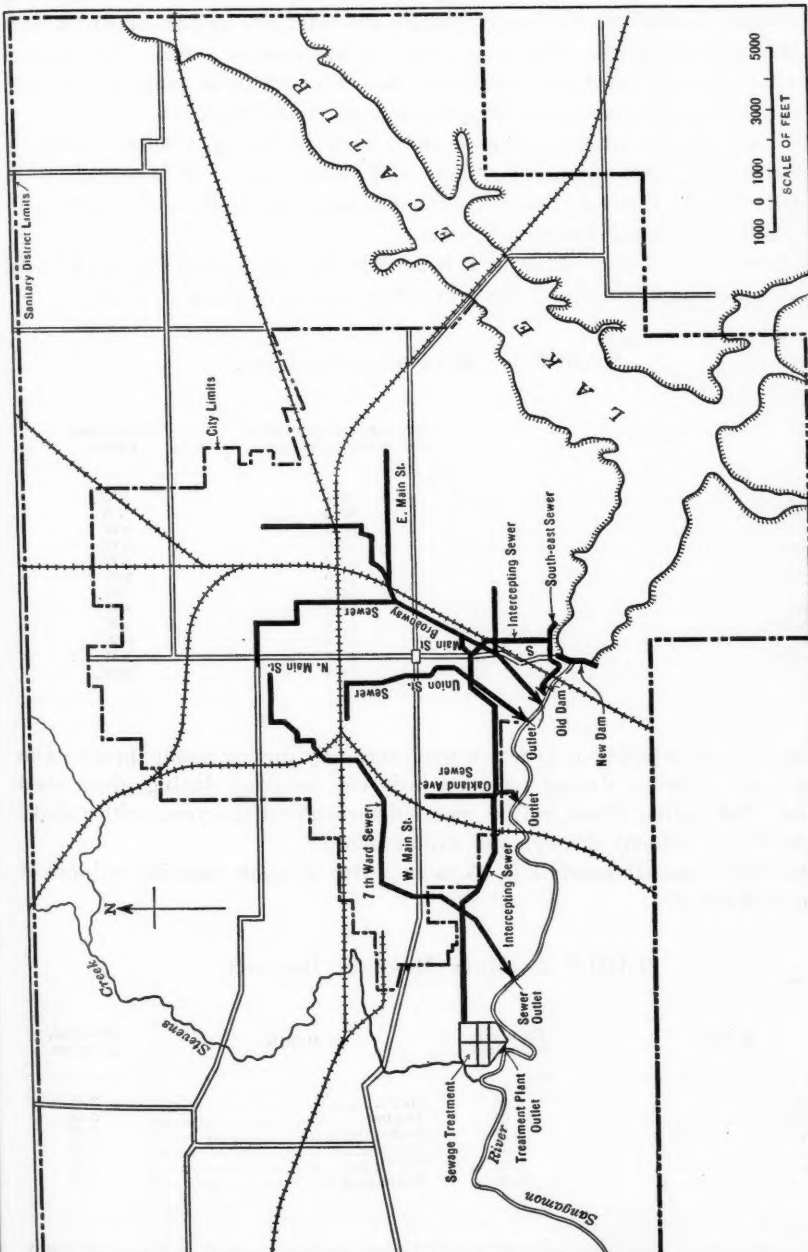


FIG. 1.—SANITARY SEWER LAYOUT, DECATUR, ILLINOIS.

The city is built mostly to the north of the river on ground 50 to 100 ft. above mean river level. This high ground is cut by several relatively deep watercourses extending northerly through the city, one or two of which have been drained and filled. The river valley is upwards of ¼ mile wide and, at present, the sewers reach the margin of the valley 20 ft. or more above mean river level. For the most part, ample drainage is available.

The city has a large variety of industries, including a starch works, a structural steel company, railway shops and many others. It is served by the Wabash, Illinois Central, Pennsylvania, Chicago and Indiana Western Railroads, and the Illinois Traction System.

*Meteorological Data.*—Decatur is located in the corn belt of Central Illinois. The average temperature and the prevailing winds are given in Table 1.

TABLE 1.—METEOROLOGICAL DATA.

| Month. | Average temperature, in degrees Fahrenheit. | Prevailing winds. |
|---|---|---|
| January | 26.5 | NW. |
| February | 27.3 | NW. |
| March | 40.9 | SW. |
| April | 52.0 | SW. |
| May | 63.3 | SW. |
| June | 72.0 | SW. |
| July | 76.3 | SW. |
| August | 74.8 | SW. |
| September | 67.8 | SW. |
| October | 55.8 | SW. |
| November | 42.2 | NW. |
| December | 30.3 | NW. |

The weather retards construction work seriously during about three months of the year. Sludge drying on open beds can be done during about eight months. Sprinkling filters can be operated throughout the year, with reduced nitrates in the effluent through the winter months.

The mean annual rainfall is 35.05 in., with a mean monthly rainfall as shown in Table 2.

TABLE 2.—MEAN MONTHLY RAINFALL.

| Month. | Rainfall, in inches | Month. | Rainfall, in inches. |
|---|---|---|---|
| January | 2.46 | July | 3.13 |
| February | 1.99 | August | 3.18 |
| March | 3.17 | September | 3.47 |
| April | 3.33 | October | 2.26 |
| May | 4.03 | November | 2.31 |
| June | 3.59 | December | 2.13 |

During wet seasons, the ground-water stands high and there is a marked infiltration into the sewers which increases the volume and lowers the tem-

perature of the sewage. Unusually heavy rains occurred in the late summer and fall of 1926 as follows:

| Month. | Rainfall, in inches. |
|---|---|
| July | 2.23 |
| August | 7.47 |
| September | 16.56 |
| October | 6.18 |
| November | 3.09 |

*The Sangamon River.*—The Sangamon River, with a drainage area of 862 sq. miles above Decatur, flows through a relatively wide valley of cultivated bottom-lands and wooded areas which provide a large flood-water storage. Floods, therefore, are of relatively low intensity and long duration. There are United States and Illinois Geological Survey gauging stations on the river at Monticello, 27 miles above Decatur, and at Riverton, 42.5 miles below Decatur, both being approximate distances by river. The maximum flood of record was at the rate of about 25 cu. ft. per sec. per sq. mile. The mean monthly flow at Decatur, estimated from the gaugings, is as follows:

| Month. | Mean monthly flow, in cubic feet per second. | Month. | Mean monthly flow, in cubic feet per second. |
|---|---|---|---|
| January | 577 | July | 354 |
| February | 1 118 | August | 277 |
| March | 916 | September | 200 |
| April | 1 047 | October | 233 |
| May | 1 133 | November | 202 |
| June | 466 | December | 142 |

Two periods of very low flow occurred as listed in Table 3.

TABLE 3.—Record of Low Flow.

| Months. | Mean Monthly Flow, in Cubic Feet per Second. | |
|---|---|---|
| | 1914. | 1920. |
| July | 12.0 | 67.5 |
| August | 5.6 | 15.8 |
| September | 10.4 | 16.9 |
| October | 6.9 | 12.4 |
| November | 8.6 | 19.6 |
| December | 13.0 | 27.6 |
| January | 12.0 | 82.8 |

## Historical Statement

*Situation in 1913.*—The condition of the Sangamon River resulting from the domestic and industrial sewage of Decatur, became sufficiently offensive in the years prior to 1913, so that complaint was made and the assistance of the

Illinois State Water Survey was secured in order to compile a complete report. During 1912 and 1913, therefore, this organization made an extensive sanitary survey of the Sangamon River with special reference to pollution by the Decatur sewages. Its report included complete chemical and bacteriological analyses of the Sangamon River for a distance of about 20 miles below Decatur and recommended the installation of settling tanks and sprinkling filters.

*Rivers and Lakes Commission Order, 1914.*—Following the Water Survey report, further hearings were held by the Rivers and Lakes Commission and the Consulting Engineer for the City, Langdon Pearse, M. Am. Soc. C. E., made a report indicating the difficulties of the problem, due in part to the industrial sewage, and outlining the investigations necessary to a proper solution. As a result, the Commission ordered the pollution of the river to be removed by January 1, 1917, thereby allowing (as it thought) sufficient time for a thorough and careful study of the problem.

*Pearse and Greeley Report of 1915.*—On June 4, 1914, the City instructed Messrs. Pearse and Greeley, as Consulting Engineers, to undertake the necessary investigation and their report was submitted on April 23, 1915. One of the major problems calling for special study was the effect on sewage treatment processes of the industrial sewages found in Decatur. Consequently, funds were appropriated for a testing station which was operated during the last half of 1914. The station was located near the mouth of the Broadway Sewer into which the wastes of the starch works, gas plant, and packing-house were discharged, in addition to the domestic sewage of 13 700 people. Unfortunately, the outbreak of the World War caused the starch works to shut down, so that tests were limited chiefly to the domestic sewage, as the other industrial wastes were very small in quantity. The station included a grit chamber, an Imhoff tank, a sprinkling filter, and a secondary settling tank. With a settling period of about 1.78 hours and the sprinkling filter dosed at an average net yield of 1 330 000 gal. per day per acre, 6 ft. 4 in. deep, the results of operation are shown in Table 4.

TABLE 4.—AVERAGE TESTING STATION RESULTS,
OCTOBER 24 TO DECEMBER 2, 1914.

| Substance. | Amount in influent, parts per million. | Amount in effluent, parts per million. | Percentage of reduction. | Percentage of increase. |
|---|---|---|---|---|
| **Imhoff Tank:** | | | | |
| Suspended matter | 221 | 108 | 52 | ...... |
| Bio-chemical oxygen demand | 171 | 167 | 2.3 | ...... |
| Organic nitrogen | 12.0 | 9.9 | 17.5 | ...... |
| Free ammonia | 14.6 | 15.6 | ...... | 6.8 |
| Nitrites | 0.12 | 0.12 | ...... | 0.0 |
| Nitrates | 0.47 | 0.47 | ...... | 0.0 |
| Oxygen consumed | 53 | 49 | 7.6 | ...... |
| **Sprinkling Filter:** | | | | |
| Suspended matter | 108 | 40 | 63 | ...... |
| Organic nitrogen | 9.9 | 1.8 | 82 | ...... |
| Free ammonia | 15.6 | 3.8 | 76 | ...... |
| Nitrites | 0.12 | 0.46 | ...... | 284 |
| Nitrates | 0.47 | 10.0 | ...... | 2 025 |
| Oxygen consumed | 49. | 18. | 62 | ...... |
| **Secondary Settling Tank:** | | | | |
| Suspended matter | 34 | 23 | 32 | ...... |

The report developed projects for intercepting sewers and sewage treatment works, recommending a high level sewer and a treatment plant comprising clarification works and sprinkling filters. It also included a careful discussion, with estimates of cost, of the activated sludge process in accordance with data then available.

*Test of Lime-Electrolytic Process, 1915.*—In 1915, the Electrolytic Sanitation Company made an offer to the City to build a small plant and to test its process in Decatur, with certain guaranties of its efficiency. The offer was accepted, the plant built, and on July 30, 1916, a 28-day test was begun. At the conclusion of the official test, the plant was given over for the City to operate.

The conclusions drawn by the City's representative may be summarized as follows: First, the process did not furnish a stable effluent except when excessive quantities of lime were used; second, the operating and upkeep costs were prohibitive; third, the method of sludge disposal was unsatisfactory; fourth, there was ample reason for differences of opinion as to whether the electrolytic action contributed materially to the efficiency of lime precipitation; and, fifth, the iron electrodes were subject to rapid corrosion—a new bank of electrodes was completely dissolved during two months' operation. On the basis of these findings, the electrolytic system was dismissed.*

*Notes on Activated Sludge, 1916.*—In April, 1916, the Consulting Engineers worked up a project for an activated sludge plant located close to the outlet of the largest existing sewer. On the basis of securing power at 1.25 cents per kw-hr., the following total annual costs were shown:

| | |
|---|---|
| 1914 Report: | |
| Sprinkling filters | $58 572 |
| Activated sludge, 2 cu. ft. air | 90 188 |
| 1916 Notes: | |
| Activated sludge, 2 cu. ft. air | $68 360 |

The lower cost in 1916 as compared with the 1914 estimates for activated sludge is partly due to a much reduced intercepting sewer cost and more favorable assumptions for excess sludge disposal.

At the request of the Decatur Association of Commerce, John W. Alvord, M. Am. Soc. C. E., was engaged to review the work done on sewage disposal and to report on the water supply. This report was submitted in May, 1917. It characterized the recommendation of the Consulting Engineers as "an obvious and conservative method of dealing with sewage"; and then suggested further investigation of two alternative projects—one comprising settling tanks and sand filters with a smaller intercepting sewer; and the other, settling tanks at each sewer outlet discharging into a so-called dilution pool formed by a low dam across the river.

From January to August, 1917, H. R. Lee, City Chemist, operated the testing station built in 1913. He reported to the City as follows:

(*a*) The Imhoff tank and sprinkling filter system will produce a stable effluent.

* *Engineering Record,* November 11, 1916.

(*b*) The maximum rate at which a 90% stability could be maintained was found to be 1 000 000 gal. per acre per day. Whether this rate is sufficiently high to make the plan attractive from the standpoint of cost, and benefits derived, is a question for the engineer to decide.

(*c*) A rate of 1 250 000 gal. per acre per day could easily be secured were the Broadway sewage diluted with an equal volume of domestic sewage.

(*d*) The Imhoff tank will remove between 50 and 60% of the suspended solids. This is a good removal for a sewage of this type.

(*e*) The Worcester nozzle was found to give the better distribution of the two types used.

These tests were conducted on sewage from the Broadway Sewer made up of about 750 000 gal. per day of starch works sewage and about 1 500 000 gal. per day of domestic sewage, and having the following approximate analysis as taken into the testing station:

| Item. | Parts per million. |
|---|---|
| Suspended matter | 413 |
| Total organic nitrogen | 87 |
| Oxygen consumed | 200 |
| Bio-chemical oxygen demand | 700 |

In reviewing this report, it was found to be unsafe to estimate the rate of application of 1917 Broadway sewage, including the starch works sewage, at more than 800 000 gal. per day per acre. During the winter of 1916-17, a citizens' committee studied methods of financing the sewage disposal works and finally it drafted and secured the enactment of the Sanitary District Act which became effective in July, 1917.* The first and only bond issue, amounting to $860 000, was favorably voted, February 24, 1920. All other expenditures have been made out of annual taxes.

In November, 1917, a committee of the City Council, reviewed the status of sewage disposal, visited about twelve sewage treatment plants in the East, and reached the following conclusions:

> (*a*) From the processes observed at the sewage filter plants visited, the Commissioners felt that sand filters and contact beds are unsuitable to conditions at Decatur because of the large area required.
>
> (*b*) That any process involving the use of lime as a precipitant would be unsuited to conditions because of the large quantity of lime required and the large volume of sludge which must be disposed of by some means.
>
> (*c*) That Imhoff tanks and sprinkling filters would be suitable for the domestic sewage and if built of sufficient size would be suitable for the combined sewage and starch factory wastes.
>
> (*d*) That activated sludge might be suitable for the disposal of Decatur sewage provided power could be obtained for less than 1 cent per kw-hr. and that some satisfactory method of dewatering, or disposing of the sludge, could be found.

The report then recommended the employment of consulting engineers and the preparation of plans and specifications.

*The Consulting Engineers' Report of 1918.*—This report was prepared shortly after the formation of the Sanitary District in 1917, to bring to date

* *Transactions*, Am. Soc. C. E., Vol. 91 (December, 1927), p. 441.

in definite form all the various prior investigations. The conclusions of this report were briefly stated, as follows:

(*a*) The seriousness of river pollution at Decatur calls for remedial measures. Decided odors are produced, particularly adjacent to the city, but which continue in lessening degree 20 miles or more below the city.

(*b*) The project requires the construction of a high-level intercepting sewer to discharge at its lower end about 19 ft. above mean low water.

(*c*) The results of operation of the testing station (1917) indicate that the mixed domestic and industrial sewage can be treated on sprinkling filters. This type of treatment plant, therefore, can be considered reliable. The rate of treatment remains to be more closely fixed on a larger scale.

(*d*) Water storage is recommended as necessary for an additional water supply. Together with river regulation, it is a helpful factor which will materially improve conditions in the river during extremely dry weather.

The report also included an appendix on subsidence resulting from coal mining. As a result, the coal rights were purchased along nearly 3 500 ft. of right of way for $7 500.

*Coal Mine Subsidence Report.*—In July, 1918, Professor L. E. Young, of the University of Illinois, submitted a report on ground subsidence under structures proposed by the Sanitary District, with recommendations, as follows:

(*a*) If possible, arrangements should be made to defer the construction of the Riverside Branch for a period of several years so that the coal under this section may be mined. Sewer construction should not begin at this point until three years after the coal directly beneath has been mined.

(*b*) On the basis of the reported market value of coal rights in this District it is recommended that a coal reservation be acquired along the line of the 1918 report. This should be a strip not less than 500 ft. wide.

(*c*) If the construction is to begin within three years, proper steps should be taken immediately to stop coal mining beneath those portions of the line which will be built.

*Notes on Starch Works Sewage, 1919.*—In the latter part of 1918, the owners of the starch works proposed to enlarge their plant for the manufacture of glucose, dextrine, and other corn products; and the City started preliminary studies for a large water impounding project. Therefore, a review of the situation with special reference to the starch works sewage was made, and analyses of this sewage from various sources were summarized, as shown in Table 5. The report concluded that, based on a maximum grind of 25 000 bushels per 24 hours, not more than double the area of sprinkling filters would be required because of the starch works sewage, as compared with domestic sewage.

During the summer of 1919, some tests of the activated sludge process on starch works sewage at the plant were undertaken by the owners. The test was run for 77 days on gluten settler waste in a wooden tank 10 ft. in diameter and 10 ft. deep, holding 5 000 gal. A test of 4 days with straight waste gave no results. The sewage frothed over the tank and no sludge or nitrifying organisms were gained. The waste was then mixed with 4% of city sewage and later with an equal volume of city water with some better results. A test

was finally run with a mixture of the waste with two volumes of city water. This mixture had a temperature between 78 and 86° Fahr. The operation was established on the following basis:

| | Time, in hours. |
|---|---|
| Filling | 1.00 |
| Aeration | 19.00 |
| Settling | 3.0 |
| Draining (dewatering) | 1.0 |

The quantity of air was not measured. The ratio of filtros plate area to tank surface was 1 to 4.75, and the air was estimated at 2.3 cu. ft. per min. per sq. ft. of tank surface. Frothing ceased after 6 to 8 hours. The analytical results of a week's operation are given in Table 6.

TABLE 5.—ANALYSES OF STARCH WORKS SEWAGE.*

| Items. | RECORDS OF VARIOUS ANALYSES, PARTS PER MILLION. | | | | |
|---|---|---|---|---|---|
| | (1). | (2). | (3). | (4). | (5). |
| Solids: | | | | | |
| Total | 290.0 | 4 860.0 | 5 866.0 | ...... | 5 000.0 |
| In solution | ...... | 3 630.0 | 2 133.0 | ...... | 3 500.0 |
| In suspension | ...... | 1 230.0 | 2 783.0 | 1 981.0 | 1 500.0 |
| Nitrogen: | | | | | |
| Total organic | 121.0 | 478.0 | 590.0 | 534.0 | 138.7 |
| Free ammonia | 3.4 | 1.7 | 1.0 | 1.3 | 1.3 |
| Nitrites | 0.0 | 0.0 | 0.02 | 0.01 | 0.0 |
| Nitrates | 2.65 | 1.50 | 0.0 | 0.75 | 1.5 |
| Oxygen consumed | 552.0 | ...... | 1 925.0 | 1 592.0 | 1 000.0 |
| Biological oxygen | 780.0 | ...... | ...... | ...... | 1 600.0 |
| Alkalinity [6] | ...... | 100.0 | ...... | 100.0 | 100.0 |
| Acidity [7] | Slightly | 520.0 | ...... | 520.0 | 520.0 |

* The crude starch works sewage contains 4 581 parts per million of total solids, of which 4 070 are in solution and 511 in suspension.

NOTES.—(1) Pearse and Greeley, April, 1915, Report sample, June 20, 1914; (2) Lees' sample, November 24, 1916; (3) Lee's sample, September 1, 1916; (4) Lee's Testing Station Report data for 1916-17; (5) typical analysis used in computations for the report; (6) alkalinity to methyl orange; and (7) acidity to phenolphthalein.

The percentage of total solids recovered as sludge was 35. After being dried and ground, this had the following analysis:

| Item. | Percentage. |
|---|---|
| Moisture | 4.3 |
| Total nitrate (N) | 7.18 |
| Potash ($K_2O$) | 0.62 |
| Phosphates ($P_2O_5$) | 2.91 |

*Dorr-Peck Experiments, 1920.*—In July, 1920, Mr. F. H. Rhodes submitted a report, based on several weeks of experiments with bottles. He stated that the Dorr-Peck process of sewage treatment was applicable to the mixed sewage at Decatur, that 4 parts per million of $Fe_2O_3$ was helpful, and that 1 000 000 gal. of sewage produced 1.2 tons of sludge, containing 6.35% of ammonia.

TABLE 6.—TESTS OF THE ACTIVATED SLUDGE PROCESS.

| Item. | RESULTS, IN PARTS PER MILLION. | |
|---|---|---|
| | Influent. | Effluent. |
| Total solids | 1 592 | 1 037 |
| Solids in solution | 1 377 | 918 |
| Solids in suspension | 215 | 119 |
| Nitrogen as: | | |
| Total organic | 125 | 23.5 |
| Ammonia | 0.2 | 55.6 |
| Nitrites | ...... | 9.1 |
| Nitrates | ...... | 0.4 |
| Oxygen consumed | 225 | 37.9 |
| Stability (methyl blue) | ...... | 71% |

*Survey of Industrial Sewage.*—From time to time since 1924, the engineers and the superintendent have investigated and reported on some of the minor industrial sewages. A small packing house, in a low-level area and with an average kill of 500 hogs and 70 cattle per week, was studied especially with reference to the operation of a small pumping station. The packing house was required to install a settling tank and grease basin on its sewer outlet. The chemical data obtained during these investigations are summarized in Table 7. All samples were 8-hour composites.

TABLE 7.—RESULTS, IN PARTS PER MILLION, OF ANALYSES OF INDUSTRIAL SEWAGES.

| Date of collection. | Source. | Suspended matter. | Total solids. | Total nitrogen. | Ammonia nitrogen. | Nitrates and nitrites. | Stability.* | Oxygen consumed. | Dissolved oxygen. | Bio-chemical oxygen demand, 5 days, 20° cent. | $SO_4$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 10/14/25 | Wabash shops | 55 | 440 | 0.2 | .... | 0.0 | 68 | 17 | 5.1 | 17 | 411 |
| 10/15/25 | Premier Malt Co., strong waste | 27 000 | 32 400 | 310 | .... | 0.0 | 0.0 | 9 840 | 0.0 | 20 000 | .... |
| 10/15/25 | Premier Malt Co., combined waste | 426 | 1 040 | 51 | .... | 0.1 | 0.0 | 278 | 0.0 | 600 | .... |
| 10/16/25 | Faries Mfg. Co. | 442 | 3 540 | 9 | 6. | 23.0 | 90 | 103 | 1.2 | 13 | 802 |
| 10/16/25 | Mueller Mfg. Co., plate room | 265 | 549 | 0.8 | .... | 0.0 | 90 | 23 | 7.8 | 50 | 125 |
| 10/16/25 | Mueller Mfg. Co., oil wash | 557 | 723 | 2.0 | .... | 35.0 | 90 | 82 | 6.2 | 308 | 314 |
| 10/16/25 | Danseizen's Packing Co.-no killing | 508 | ...... | 2.6 | 16. | .... | .... | 207 | .... | ...... | .... |
| 10/16/25 | Danseizen's Packing Co.-killing | 724 | 3 550 | 440. | .... | 1.5 | 0.0 | 674 | 0.0 | 1 393 | .... |
| 10/20/25 | Young's Packing Co. | 1 660 | 4 190 | 204 | .... | 1.0 | 0.0 | 708 | 0.0 | 693 | .... |

* Percentage of stability to methylene blue.

NOTE.—Population equivalent calculated from bio-chemical oxygen demand and volume of waste:

| | |
|---|---|
| Wabash Shops | Negligible |
| Premier Malt Co. | 3 000 |
| Faries Mfg. Co. | Negligible |
| Mueller Mfg. Co. | Negligible |
| Danseizen's Packing Co. | 7 000 |
| Young's Packing Co. (estimated) | 3 000 |
| Total | 13 000 |

With the completion of the water impounding project and the intercepting sewer, it became advisable to make a comprehensive general sewer plan for the entire city providing for new and relief sewers. The city was expanding and the newly built-up areas required sewerage. Many of the existing sewers were overloaded and needed relief. The City and District officials desired to have such sewers properly related to the sewage disposal and river improvement works. A general plan was prepared showing about six major sewer projects, with a total estimated cost of \$2 175 200. One of the projects, amounting to about \$360 000, was completed in 1927; and plans and specifications were ready for taking bids on another estimated to cost more than \$600 000.

As a result of all this effort, intercepting sewers and sewage treatment works have been completed and put into operation with an equivalent population capacity, under conditions at Decatur, of about 150 000. The owners of the starch works are completing process adjustments reducing the quantity and strength of its sewage to an approximate population equivalent of 1 000 per 1 000 bushels of grind. Construction work was first started in 1919, when the upper section of the intercepting sewer was let; and was completed in October, 1927, when the pre-aeration plant was put into operation.

## Design Data

*Sources.*—The progressive design and construction of the Decatur sewage disposal projects over so many years, has made it possible to secure some design data from operating records, as, for instance, sewage quantities and characteristics at the treatment plant. In the first place (1914), weirs were built at the main sewer outlets and the rates of flow measured. As already noted, much design data came from the operation of the testing stations. The remainder was secured through the usual routine of local investigation.

*Population.*—The first population forecast and distribution of population over the city was made in 1915, and was reviewed in 1919 and 1925. The basis of design of works easily enlarged was 60 000, and of intercepting sewers and structures not easily enlarged, 120 000. An additional allowance was made for the sewage of major industries.

*Quantities of Sewage.*—The capacity of the intercepting sewer varies for different sections, as shown on Fig. 2. In the upper section where separate sewers are to be built, the capacity flowing full is for sewage at the rate of 1 000 gal. per capita per 24 hours from 20 000 people. This allowance includes capacity for a considerable industrial area. Then follows a short section taking combined sewage from about 250 acres located above the dam with a capacity of 5 750 gal. per capita per 24 hours from 28 700 population. There is an overflow just below the dam, and the capacity of the next section, which is largely in tunnel, is for sewage at the rate of 730 gal. per capita per 24 hours from 28 700 people. The next section, which is through a built-up part of the city, provides capacity for 950 gal. per capita per 24 hours from 68 000 people. The final section built largely in open country and, therefore, quite easily duplicated, has a capacity of 40 700 000 gal. per day equivalent to 340 gal. per capita from 120 000 people.

The first installation of sewage treatment works (Imhoff tanks, sprinkling filters, and appurtenances) was given a capacity for 60 000 people and 7 960 000 gal. per day, equivalent to 133 gal. per capita per 24 hours. This included 1 250 000 gal. per day of starch works sewage, equivalent to about 21 gal. per capita per 24 hours. When the additions to the sewage treatment plant were designed in 1926, the rates of flow (see Table 8) were estimated largely on the basis of meter records for about two years at the treatment plant.

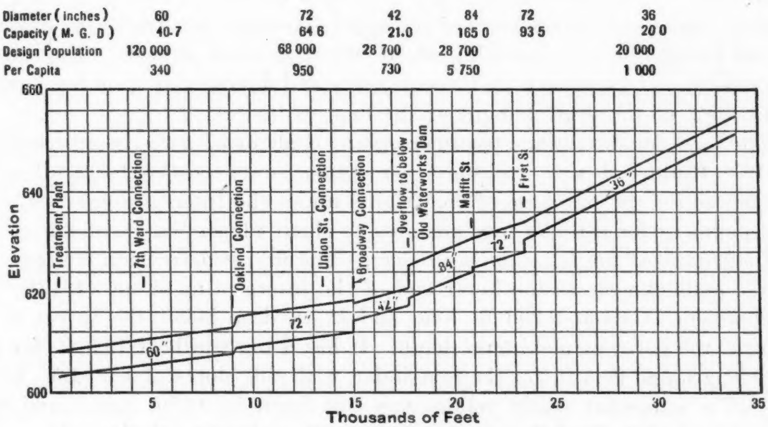


FIG. 2.—PROFILE OF INTERCEPTING SEWERS, DECATUR, ILL.

Including industrial sewage, the average dry season rate of flow is 167 gal. per capita per 24 hours from 60 000 people. It has been difficult to estimate the quantity of sewage from the starch works because of the rapid increase in the grind and the process adjustments resulting in changing quantities of sewage per bushel of grind.

TABLE 8.—RATES OF FLOW ESTIMATED IN 1926.

| Item. | MILLION GALLONS PER 24 HOURS. | |
|---|---|---|
| | Wet season. | Dry season. |
| Average | 14.0 | 10.0 |
| Maximum | 16.0 | .... |
| Minimum | .... | 5.2 |

*Sewage Characteristics.*—A determination of the strength and character of Decatur sewage has been a most difficult element in the design. The domestic sewage is mixed with a variable industrial sewage from the starch works. This industrial sewage has increased with the grind from a population equivalent of 75 000 more or less during the period of 1914 to 1917, to one of more than 300 000 in 1926. In addition, the grind and the sewage vary with business conditions, the grind having recently ranged in a single

year from 20 000 to 45 000 bushels per day. Recently, the owners of the starch works, under the direction of Dr. Edward Bartow, have undertaken process adjustments to try and recover products which, until recently, were lost to the sewer, with a marked reduction in the strength of their sewage. The sewage first increased in strength with the growth at the starch works; and then decreased with the reduced waste.

## Construction Projects

*General.*—Construction work has proceeded almost continuously from 1919 to 1927 with a total expenditure chargeable to construction, including engineering, of $1 666 783, and involving about fifteen general projects. Engineering supervision and inspection on these projects and during this time has equalled about 6.0% of the total amount of the final estimates.

The land for the sewage treatment plant is about 57 acres in area and cost $7 419. Rights of way for the intercepting sewer cost $11 702, of which $7 500 was for the purchase of coal rights under the intercepting sewer. The total payment for right of way amounts to about $1 per lin. ft. of sewer.

*Embankment for Intercepting Sewer.*—The first construction contract was for the embankment at the lower end of the intercepting sewer (1919). The embankment is about 2 900 ft. long and 12 ft. deep below the invert of the sewer at the point of maximum depth. It has a top width of 19.5 ft. for part of its length and 26.5 ft. for the remainder, with side slopes of 1 on 1.5. It was built of a somewhat sandy yellow clay put down in 12-in. layers and compacted by the travel of the teams. It was allowed to stand about 10 months before further use. After 5 years of service, there have been no signs of settlement. The sides have been planted with hay, clover, etc., and there has been no need of repairing them. The original contract comprised about 35 000 cu. yd. of embankment at $0.34 per cu. yd. in place. All the material was taken from a borrow-pit near the right of way.

*Intercepting Sewers.*—The intercepting sewer built by the Sanitary District is 18 317 ft. long and cost $598 000, or an average of $32.60 per ft. In the three contracts under which it was built, alternate bids were taken on several types of construction and the least expensive type was selected with results as follows:

| Type. | Approximate length, in feet. |
|---|---|
| Monolithic concrete | 5 700 |
| Segmental block | 8 700 |
| Reinforced concrete pipe | 3 600 |

*Sewage Treatment Works.*—The sewage treatment works were started in 1921 with the construction of the treatment plant by-pass. This was a 48-in. concrete sewer, 1 131 ft. long, including an overflow chamber, an outfall structure, and appurtenances. The sewer cost $20.20 per ft.

Other construction projects at the sewage treatment plant were a roadway, 3 100 ft. long; an elevated tank with a capacity of 50 000 gal.; a ground-water supply; a sludge locomotive; and an electric power line. The total amount spent in connection with these various projects was $653 143, including engi-

neering. Except for a rather liberal sludge capacity in the Imhoff tanks (3.03 cu. ft. per capita), the works so built have a capacity for a strong domestic sewage from about 60 000 people, indicating a cost of $10.92 per capita.

In the spring of 1925, a small pre-aeration testing station was put into operation; and during 1926 and the first half of 1927, the sewage treatment plant was enlarged by the addition of pre-aeration tanks and appurtenances.

*South Main Street Pumping Station.*—A small low-level area of about 175 acres is served by the South Main Street Pumping Station. The area includes two small packing houses with an average kill of about 500 hogs and 70 cattle per week. The packing-house sewage passes through a small grease basin before reaching the pumping station. The pumping station contains bar screens with $\frac{5}{8}$-in. clear openings and two 4-in. vertical centrifugal pumps with motors operated by floats. The station is automatic in operation and is visited once a day for inspection, cleaning screens, and oiling. This station cost $12 454 to build, and has an installed pump capacity of 1 000 000 gal. per day.

*Pre-Aeration Testing Station.*—During February and March, 1925, a small testing station (Fig. 3) was built to study the effect of partial or pre-aeration on sprinkling filter rates. The major items of construction are briefly described as follows:

(*a*) An aeration tank 12 ft. in diameter, 10.0 ft. liquid depth, and containing 6 570 gal. At 2 hours' displacement, this tank had a capacity for 78 000 gal. of sewage per 24 hours. It was fitted with a surface aerator or agitator of the Simplex type. The agitator wheel was 36 in. in diameter and operated by a 1-h.p. motor at 80 rev. per min.

(*b*) Originally, there were two settling tanks, each 5 ft. in diameter and 9 ft. liquid depth. At a sewage flow of 78 800 gal. per 24 hours the displacement period was 0.63 hours, with an area for about 2 000 gal. per sq. ft. per 24 hours, with both tanks in operation. These rates were found to be too high and a single tank, 10 ft. in diameter and 10 ft. liquid depth, was added. This provided a displacement period of 1.5 hours, with an area for about 670 gal. per sq. ft. per 24 hours, with all three tanks in service.

(*c*) The sludge re-aeration tank had a capacity of 810 gal. With 10% return sludge (8 000 gal. per 24 hours), the aeration period was about 2.5 hours.

(*d*) The sprinkling filter was 14 ft. in diameter and 6 ft. 4 in. deep, with one Taylor nozzle. It was filled with 1 to 2-in. stone from the large sprinkling filter; and dosed from a small tank controlled at first by a motor-driven butterfly valve, and, later, by an automatic siphon.

(*e*) In addition to the aeration tank, settling tanks, re-aeration tank, and sprinkling filter, the testing station included a blower, an air meter, a small pump to serve the sprinkling filter, a number of orifice boxes, and other appurtenances.

It took about a month to build the station and the costs were approximately as follows:

| | |
|---|---|
| Materials ........................... | $3 230 |
| Labor ................................ | 800 |
| Engineering ......................... | 652 |
| Total.............................. | $4 682 |

*Covering Grit Chambers, Conduits, Gas Vents, Etc.*—The starch works sewage at Decatur contains very substantial quantities of sulfur dioxide and soluble organic matter which, with the domestic sewage, decomposes to form

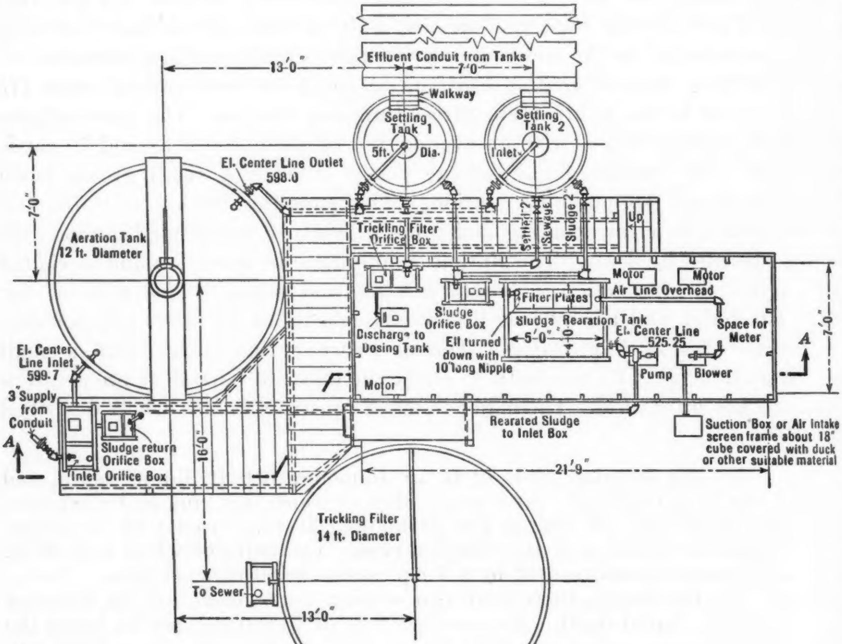


FIG. 3.—GENERAL PLAN, TESTING STATION, DECATUR, ILL.

hydrogen sulfide. This decomposition is greatly accelerated by the relatively high temperature. To reduce such odors, concrete covers were built during the summer of 1926 over the grit chamber and the conduits and gas vents of

the Imhoff tanks. The spaces under the covers of the grit chamber and conduits were connected by pipes to a suction fan which delivered the confined air and gases to a small brick furnace in which they were burned with the gases from the digesting sludge. The covers over the gas vents are shown in Fig. 4. The cost of this work, including engineering, was about $15 000.

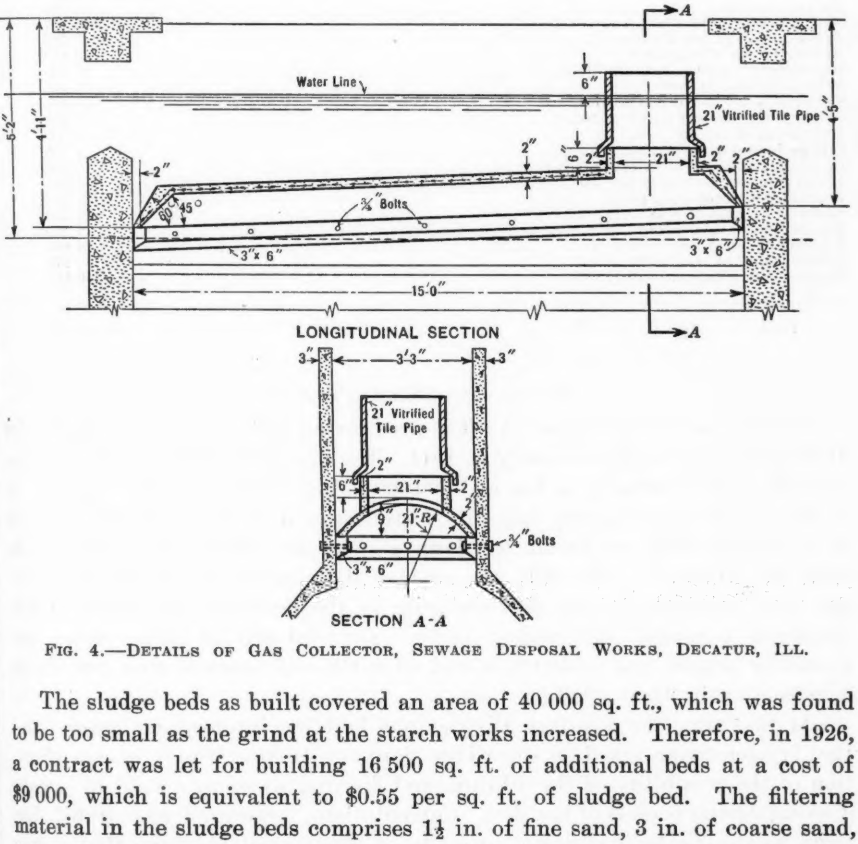


Fig. 4.—Details of Gas Collector, Sewage Disposal Works, Decatur, Ill.

The sludge beds as built covered an area of 40 000 sq. ft., which was found to be too small as the grind at the starch works increased. Therefore, in 1926, a contract was let for building 16 500 sq. ft. of additional beds at a cost of $9 000, which is equivalent to $0.55 per sq. ft. of sludge bed. The filtering material in the sludge beds comprises 1½ in. of fine sand, 3 in. of coarse sand, 4 in. of roofing gravel (¼ to ⅜ in. in size), and below this a layer of graded broken stone or gravel up to 2-in. size about the under-drains.

During the fall of 1926 and until the autumn of 1927, a pre-aeration plant was built. This plant comprises six aeration tanks and two settling tanks of the Dorr type, all with a rated capacity of 10 000 000 gal. per day.

As nearly as may be determined from the records of the Sanitary District, the total expenditures for construction projects have amounted to $1 666 783 (see Table 9). Active construction work started just after the World War in 1919 and was completed in 1927, covering nine construction seasons. This is a total cost of $33.30 per capita, based on 50 000 population for the period of construction.

TABLE 9.—SUMMARY OF EXPENDITURES FOR CONSTRUCTION PROJECTS.

| Item. | Amount. |
|---|---|
| Embankment for intercepting sewer, 1919 | $53 620 |
| Intercepting sewer, Contract 2 | 135 131 |
| Intercepting sewer, Contract 3 | 329 205 |
| Intercepting sewer, Contract 4 | 134 365 |
| By-pass sewer | 22 894 |
| Sewage treatment plant, Contract 1 | 570 707 |
| Pre-aeration plant, Contract 5 | 142 785 |
| Blower house, Contract 6 | 44 663 |
| Equipment for pre-aeration plant | 27 832 |
| Wrecking dosing tank | 1 100 |
| Seventh Ward sewer connection | 19 681 |
| South Main Street pumping station | 12 454 |
| Extension of sludge beds | 9 000 |
| Sludge locomotive | 1 805 |
| Gas collecting and burning apparatus | 15 000 |
| Electric power transmission line | 5 255 |
| Roadway to treatment plant | 7 724 |
| Land and rights-of-way | 19 122 |
| Equipment of laboratory | 2 820 |
| Treatment plant water supply | 10 639 |
| Landscaping at treatment plant | 971 |
| Engineering (approximate), 1927 | 100 000 |
| Total | $1 666 783 |

## WATER IMPOUNDING PROJECT

*Relation to Sewage Disposal.*—The need for an additional water supply for Decatur became evident as early as 1914. The unregulated flow in the Sangamon River is frequently as low as 5 000 000 gal. per day for several months at a time. On one occasion only the fortunate and unexplained blowing up of a channel dam up stream let down a sufficient volume of water to tide over the situation. By 1920, the average daily water consumption of the city was 5 540 000 gal. per day, exclusive of the needs of the Staley Manufacturing Company for cooling water. Consequently, a large water impounding project was undertaken and an earth and concrete dam put under construction in June, 1920.

At that time the Sanitary District was building intercepting sewers, and the Trustees were asked to contribute Sanitary District funds in consideration of the possibility of the diluting and flushing water as an aid to sewage disposal during periods of low flow. Computations, however, demonstrated that there would not be a sufficient quantity of water available from the storage reservoir to supply enough oxygen for the sewage and no appropriations were made for this purpose.

Taking 500 parts per million as the approximate average oxygen demand of the sewage, a sewage flow at the rate of 10 000 000 gal. per day would require about 42 000 lb. of oxygen per 24 hours. Assuming 5.0 parts per million of dissolved oxygen available from the reservoir water and no measurable flow in the river, it would require a discharge from the reservoir at the rate of 1 000 000 000 gal. per day. Obviously, nothing like this rate of discharge was possible except for very short periods, as the safe yield of the impounding project was estimated at about 35 000 000 gal. per day. Nevertheless, the river near the city has been freshened by the occasional discharge of reservoir water as indicated in Table 10.

*Brief Description.*—Lake Decatur is formed by an earth and concrete dam across the valley of the Sangamon River. The total length of the dam is 1 715 ft. At the north end is an earth embankment, 510 ft. long. The center section, a solid concrete spillway, 525 ft. long, is 28.5 ft. high and has a width at the base of 72 ft., including the down-stream concrete apron. There is an up-stream clay apron, 28 ft. wide, making the total width of the base of the dam 100 ft. A movable crest is built on top of the spillway by which the water level can be raised 2.5 ft. up to Elevation 612.5.

TABLE 10.—River Flushing with Reservoir Water.

| Date. | Duration of flushing, in hours. | Approximate Rate of Flushing. | |
|---|---|---|---|
| | | In million gallons per day. | In cubic feet per second. |
| 1925: | | | |
| June 6 | 22 | 44 | 67 |
| July 18 | 24 | 44 | 67 |
| July 28 | 24 | 44 | 67 |
| August 6 | 22 | 51 | 79 |
| August 18 | 22 | 51 | 79 |
| August 25 | 21 | 51 | 79 |
| September 9 | 24 | 51 | 79 |
| 1926: | | | |
| July 2 | 26 | 73 | 112 |
| July 16 to 30 | Intermittent | 177 | 272 |
| August 16 to 20 | " | 177 | 272 |

Extending under 1 500 ft. of the length of the dam is a line of sheet-piling largely of steel, but with wood sheeting at either end. Under the concrete part of the dam (see Fig. 5), the steel sheet-piling extends down about 30 ft. to a firm connection with a layer of hardpan glacial till. At the north end of the spillway, a gate chamber has been provided so that 1 000-h.p. water turbines can be installed if desired.

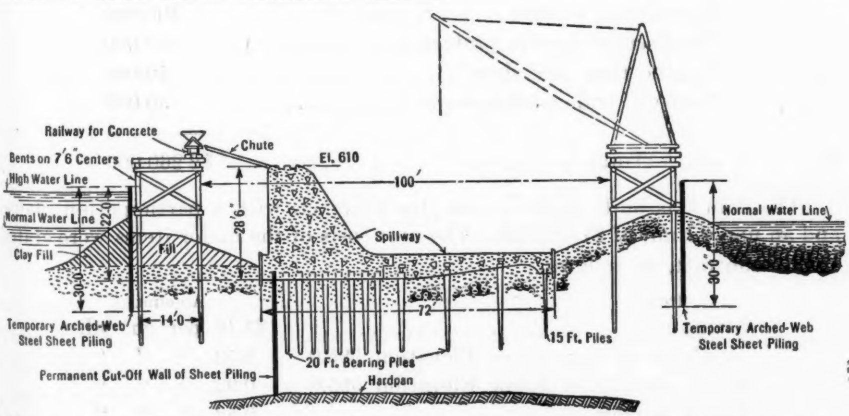


Fig. 5.—Cross-Section of Coffer-Dam for Concrete Spillway of Dam, Decatur, Ill.

The dam was built on a sandy gravelly formation underlying the valley soil and overlying the Wisconsin and Illinoisan tills. The Wisconsin till

(or drift) is of terminal moraine origin and the Illinoisan of ground moraine structure. Thus, the Illinoisan till is very hard and dense with a maximum content of impervious clay. For the most part the steel sheeting was driven into this formation effecting a very good cut-off.

The reservoir back of the dam covers about 3 000 acres and extends up stream about 12 miles, with an average width of somewhat less than ½ mile. The volume of the reservoir is from 6 000 000 000 to 8 000 000 000 gal. and the average depth 7.5 ft. About one-fourth of the area was timbered and had to be cleared. Nearly fifty buildings were removed and 17 miles of marginal and cross-highways were raised, relocated, and protected with paving.

*Method of Financing.*—The water-impounding project cost more than $2 000 000. The City had available the proceeds of two bond issues amounting to somewhat more than $600 000. Money from annual taxes was appropriated for construction purposes, amounting to more than $250 000. It was necessary, therefore, to raise about $1 300 000 in some other way. The plan adopted was to form a local water company of Decatur citizens that was to issue stock and notes secured by the reservoir land and by a contract with the City to pay the water company enough to provide a 7% rate of interest and to retire the stock in 16 years. This stock in the amount of $1 000 000 was over-subscribed in a short campaign, and the necessary funds were secured. By other subscriptions and loans this amount was raised to more than $1 300 000.

The cost of the water impounding project may be summarized, approximately, as follows:

| Item. | Amount. |
|---|---|
| Land for dam | $30 000 |
| Dam | 815 000 |
| Reservoir land | 610 000 |
| Clearing | 115 000 |
| Roads and bridges | 400 000 |
| Road embankment protection | 200 000 |
| Engineering for dam | 40 000 |
| Surveys and miscellaneous | 50 000 |
| Total | $2 260 000 |

The dam was built under a cost-plus contract, with a participation clause and an upper limit of $1 025 000. The unit costs of the major items were stated in the contract, as follows:

| Item. | Amount. |
|---|---|
| Stripping | $1.10 per cu. yd. |
| Earth excavation above Elevation 595.0 | 0.52 " " " |
| Earth excavation below Elevation 595.0 | 0.92 " " " |
| Earth borrow | 0.52 " " " |
| Rock excavation | 3.00 " " " |
| Earth embankment, Class A | 0.17 " " " |
| Earth embankment, Class B | 2.30 " " " |

| Item. | Amount. |
|---|---|
| Placing loam | $1.70 per cu. yd. |
| Seeding | 0.06 " sq. yd. |
| Drainage for slope paving | 0.23 " lin. ft. |
| Slope paving | 1.55 " sq. yd. |
| Steel sheet-piling, Class A | 0.07 " lb. |
| Steel sheet-piling, Class B | 0.06 " " |
| Wooden sheet-piling | 1.05 " sq. ft. |
| Concrete in spillway | 12.20 " cu. yd. |
| Concrete in gate setting | 17.00 " " " |
| Concrete in apron | 9.20 " " " |
| Concrete in toe-wall | 15.00 " " " |
| Concrete in abutments | 13.70 " " " |
| Concrete in conduit | 25.00 " " " |
| Reinforcing steel | 0.06 " lb. |

## Notes on Starch Works

*Growth of Starch Works.*—In 1912, the cornstarch factory began operating. The capacity of the plant at that time was about 10 000 bushels of corn per 24 hours which, by 1915, had increased to 15 000 bushels. Then, due to the World War, the plant was practically closed until 1917, when regular operation was resumed. Since that time it has operated almost continuously with a progressively increasing grind to as much as 45 000 bushels per 24 hours. The products of manufacture have lately been diversified, so that at present various types of corn starch, gluten meal, dextrine, glucose, corn sugar, corn oil, soy bean oil, and soy bean meal are produced. The Company employs about 2 000 hands when operating near the rated capacity.

The rather phenomenal growth was entirely unanticipated, even by the officials of the industry itself. In 1918 when plans for the Decatur sewage treatment plant were being formulated, the grind was less than 15 000 bushels of corn per 24 hours. The Company fully stated its plans for the future, which anticipated a maximum daily grind of 25 000 bushels and the erection of a plant for converting wet starch into glucose and corn sugar. The glucose plant was built to take advantage of the market prices of dry starch and glucose so as to secure the best gross revenue. This plant did not in itself increase the corn-grinding capacity of the starch plant, nor was it anticipated that it would. In reality, however, it has been the cause of doubling the anticipated corn grind, through better adjustment to the market and a consequent growth of business.

*Plant Processes and Wastes.*—Briefly the process of manufacture is as follows: (*a*) Steeping the corn in a dilute sulfur dioxide solution for a number of hours; (*b*) crushing the steeped corn and separating the germ from the hull; (*c*) grinding the hull, and separating the husks from the ground meal; (*d*) separating the starch and gluten by sedimentation and flotation; (*e*) washing and drying the starch and gluten; and (*f*) converting wet starch to glucose and sugar.

A certain amount of liquid waste results from each operation. In some cases the wastes are re-used in the plant for carrying purposes and thus return

into the process; in others, the liquids are evaporated and the concentrated materials are mixed and dried for the market. The sequence of the various processes is indicated in Fig. 6.

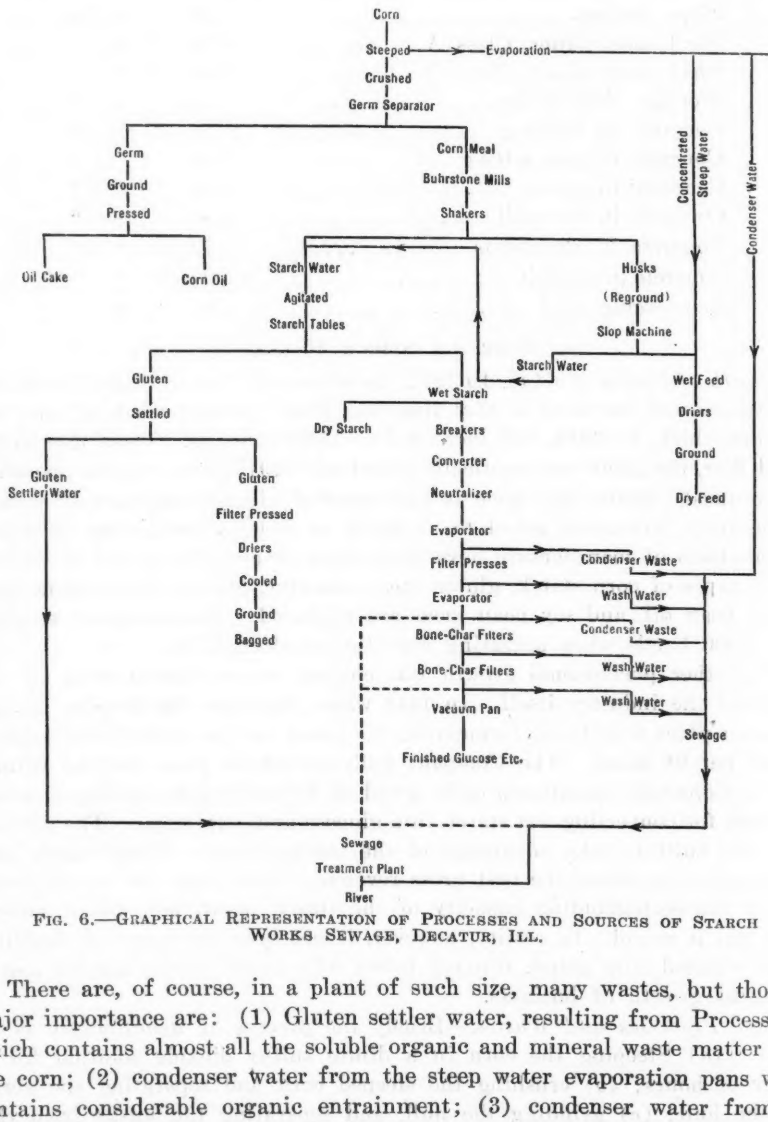


FIG. 6.—GRAPHICAL REPRESENTATION OF PROCESSES AND SOURCES OF STARCH WORKS SEWAGE, DECATUR, ILL.

There are, of course, in a plant of such size, many wastes, but those of major importance are: (1) Gluten settler water, resulting from Process (*d*), which contains almost all the soluble organic and mineral waste matter from the corn; (2) condenser water from the steep water evaporation pans which contains considerable organic entrainment; (3) condenser water from the glucose plant which is relatively free from entrainment; and (4) wash water from cleaning out condenser pans, bone filters, filter cloths, etc.

The maximum quantities of sewage expected from the starch plant was 50 gal. of strong sewage and 200 gal. of condenser water per bushel of corn ground. These were the quantities on which the design of the original

sewage treatment plant was predicated, with the expectation that the condenser water would be by-passed direct to the river if desired.

The sewage treatment plant was placed in operation in May, 1924, and during its initial operation the total quantity of mixed sewage was between 15 000 000 and 17 000 000 gal. per 24 hours. This comprised about 5 000 000 gal. per day of domestic sewage from the city and from 10 000 000 to 12 000 000 gal. per day of industrial sewage from the starch works. Investigation showed (*a*) that the starch works were grinding about 35 000 instead of the anticipated 25 000 bushels of corn per day; (*b*) that the Company's sewers were arranged so that the 6 000 000 to 10 000 000 gal. of condenser water could not be separated from the strong sewage for diversion into the river; and (*c*) that the private 24-in. sewer, through which the condenser water was to have been diverted, was of insufficient capacity.

Investigations and analyses of the glucose condenser water showed that it was practically of the same character as the raw water before entering the condenser, except for occasional entrainments of glucose or sugar. By careful operation of the evaporation pans it was thought that this entrainment could be reduced to a negligible minimum. The Company, therefore, was allowed to construct a conduit for returning this condenser water to the impounding reservoir. This was completed and placed in operation in July, 1925, and relieved the sewage treatment plant of from 6 000 000 to 10 000 000 gal. per day of relatively hot condenser water, the volume depending on the season of the year and the temperature of the cooling water from the lake. This diversion reduced the volume of sewage received at the treatment plant to about 11 000 000 gal. per day, 50% of which was from the starch works and the remaining 50% from the city.

*Characteristics of Mixed Sewage.*—The five-day bio-chemical oxygen demand of the week-day city sewage, when the starch works is not in operation, is about 125 parts per million. The average monthly bio-chemical oxygen demand of the mixed sewage, including that from the starch works, varies directly with the corn grind and inversely with the volume of sewage, and has ranged from 500 to 800 parts per million in dry weather.

The temperature of the mixed sewage as received at the treatment plant has varied from 70 to 104° Fahr. during winter and summer, respectively. These very warm temperatures caused rapid decomposition of the mixed sewage, so that when it reached the sewage plant, hydrogen sulfide odors resulted. These odors were intensified after the condenser water was removed from the sewers, due to the greater concentration of the mixed sewage so that paint on houses a mile from the plant was discolored.

*Population Equivalent and Its Reduction.*—Analyses of the city sewage and the starch works sewage at this time indicated a total population equivalent of about 350 000 for the mixed sewage as received at the treatment plant, of which about 300 000 was contributed by the starch works. These figures were obtained by dividing the total pounds per 24 hours of 5-day bio-chemical oxygen demand by 0.17 lb. as the per capita contribution of

5-day bio-chemical oxygen demand.* This factor seems to apply satisfactorily to the Decatur domestic sewage population equivalent when the starch works is shut down. The estimated human population connected to the sewers is about 40 000 and the minor industrial population equivalent, exclusive of the starch waste, is between 10 000 and 15 000. (See "Survey of Industrial Sewage," page 2247.)

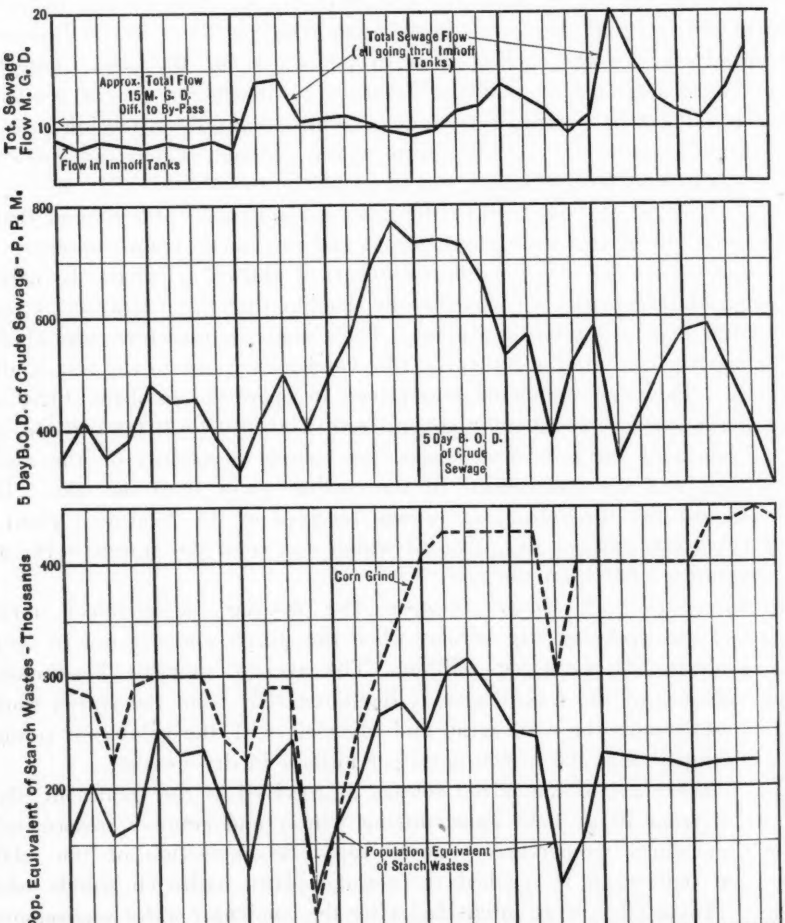


FIG. 7.—STARCH WORKS DATA.

On Fig. 7 is shown the quantities of sewage, in million gallons per day, the 5-day bio-chemical oxygen demand, in parts per million, the corn grind, in bushels per 24 hours, and the population equivalent of the starch works

* Calculated from average of eight cities, U. S. Public Health Service *Bulletin 132*, pp. 35-111.

sewage for 1924-27. The population equivalent of the starch works sewage was calculated by subtracting 50 000 from the total population equivalent as determined from the regular daily analyses of composite samples of the mixed sewage received at the sewage treatment plant.

These curves show that the Company has improved its operation and decreased its losses as indicated by the maximum population equivalent of 370 000 in February, 1926, and the average population equivalent of 275 000 for January, February, and March, 1927, when the grind of corn was higher than in February, 1926. Further reductions are expected in the near future that will bring the population equivalent of the starch works sewage to less than 100 000, thus greatly relieving the burden now imposed on the city sewage treatment plant. At present (November, 1927), the population equivalent of the starch works sewage is about 260 000.

## Sewage Treatment Works, First Installation

*General Arrangement.*—In 1922, the Board of Trustees authorized the preparation of plans and specifications for the sewage treatment plant comprising settling tanks of the Imhoff type, sprinkling filters, and appurtenances. The rated capacity of the plant was 60 000 people and an average dry-season sewage flow at the rate of 8 000 000 gal. per day. It was expected that a higher rate of flow might occur for several weeks at a time during wet seasons. Included in the quantity of 8 000 000 gal. per day was an allowance of 1 250 000 gal. per day for sewage from the starch works.

The sewage treatment plant is built on 57 acres of low land, about 2.5 miles from the business center. First-class houses are within 2 000 ft. and closer lots are being steadily developed. The general plan of the first construction is shown in Fig. 8. The site is surrounded by a flood dike with its top 19 ft. above mean low water in the river. The high water of 1926 just reached this elevation. The general plan provided for doubling the capacity or for an ultimate capacity of 120 000. The first installation included only 3.0 acres of sprinkling filter, or one-half the area needed for the stated capacity of 60 000 and the 1 250 000 gal. per day of starch works sewage.

The elevations of the principal structures were determined as shown in Table 11.

Hydraulic gradients, velocities, and elevations were computed for an average sewage flow of 8 000 000 gal. per day with a minimum flow of 5 400 000 gal. and a maximum of 44 200 000 gal. per 24 hours. Shapes and sizes were designed to maintain approximately the following velocities:

| Item. | Velocity, in feet per second. |
|---|---|
| Grit chamber | 1.0 |
| Conduit to settling tanks | 1.0 |
| Settling tank influent conduits | 1.0 |
| Settling tank effluent conduits | 0.5 |
| Sprinkling filter collecting conduits | 1.5 |
| Main effluent conduits | 2.0 |

The total loss of head is thus 20.91 ft., of which 1 ft. was included ahead of the settling tanks as a contingent allowance to permit installation of sewage treatment devices not developed at the time of design.

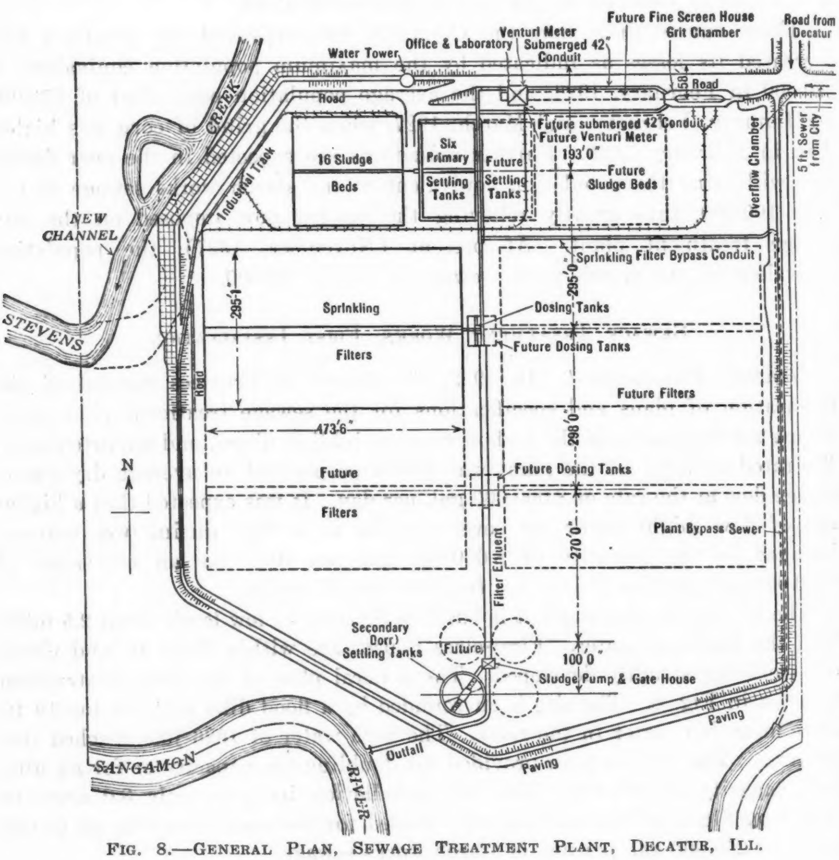


FIG. 8.—GENERAL PLAN, SEWAGE TREATMENT PLANT, DECATUR, ILL.

*Parts of the Plant.*—The principal parts of the plant comprising twenty-one items, are listed as follows:

1.—Plant by-pass sewer.
2.—Connecting sewers.
3.—Coarse screen.
4.—Grit chamber.
5.—Venturi meter and house.
6.—Imhoff tanks.
7.—Sludge beds.
8.—Dosing tanks.
9.—Sprinkling filters.
10.—Final settling tanks.
11.—Outlet to river.
12.—Filter by-pass.
13.—Final settling tank by-pass.
14.—Water supply.
15.—Lighting.
16.—Industrial railway.
17.—Drainage system.
18.—Office building and laboratory.
19.—Flood protection.
20.—Planting.
21.—Roadways.

The grit chamber* comprises three flowing-through conduits, each 75 ft. long. Two of these conduits are 8.0 ft., and the other, 10.0 ft., wide, with 1$\frac{5}{8}$-in. clear openings. The velocity and depth of flow in the conduits are maintained by a proportional orifice at the outlet end. The floor of the grit chamber stands about 8$\frac{3}{4}$ ft. in the clear above the ground level, so that sludge cars on industrial tracks are run in under the chambers for the remaining grit. Each flowing-through compartment has three 18-in. gate-valves controlling openings in the floor through which grit is dumped into the cars.

TABLE 11.

| Item. | Elevation, in feet. |
|---|---|
| Invert of sewer at entrance to plant | 602.96 |
| Sewage surface at entrance to plant | 604.41 |
| Sewage surface at entrance to settling tanks | 603.17 |
| Average elevation of sewage surface in settling tanks | 602.61 |
| Sewage surface at outlet of settling tanks | 602.13 |
| Sewage surface at first dosing tank | 602.12 |
| High sewage mark in dosing tank | 601.90 |
| Low sewage mark in dosing tank | 595.90 |
| Surface of filter stone at near nozzle | 595.10 |
| Surface of filter stone at far nozzle | 594.30 |
| Invert of under-drains at northwest corner of filter | 588.05 |
| Invert of main collector near first dosing tank | 584.78 |
| Sewage surface in main collector near first dosing tank | 586.88 |
| Invert of main conduit at final settling tanks | 584.32 |
| Sewage surface in final settling tank | 586.08 |
| Water surface in Sangamon River, August, 1921 | 585.4 |
| Low water | 583.5 |
| High water | 598.5 |
| Top of flood dike | 602.5 |
| Center line of settling tanks sludge outlets | 598.08 |
| Invert of sludge channel at settling tanks | 597.67 |
| Sand surface in sludge beds | 592.18 |
| Invert of 12-in. main collector at settling tank end of sludge beds | 588.76 |

A 42-in. cast-iron pressure conduit extends from the grit chamber to the settling tanks. This is designed for a minimum velocity of 1.0 ft. per sec.; and, later, will have to be duplicated. On its way the conduit passes through the basement of the office building where it includes a 42-in. Venturi meter tube, the register and recorder of which are set in the office above.

The settling tanks are of the two-story or Imhoff type. The first installation comprises six units, each 27 ft. 10 in. wide and 96 ft. long, inside the walls at the top. The liquid depth is 27.0 ft. and the free-board is 24 in. In each tank are two flowing-through or settling compartments. There are three gas vents in each of the three tanks having a total width of 9.0 ft. equivalent to 31.1% of the total tank surface.

The six settling tanks provide the following capacities:

| Item. | Cubic feet. |
|---|---|
| Settling | 63 700 |
| Sludge | 181 900 |
| Neutral zone }<br>Scum } | 83 000 |

* *Transactions*, Am. Soc. C. E., Vol. 91 (December, 1927), p. 535.

These volumes provide a displacement period of 1.44 hours in the settling compartments when the flow is 8 000 000 gal. per day and 3.03 cu. ft. per capita in the sludge compartment.

There are two conduits at each end of the settling tanks and a single conduit along the outside of each of the two end tanks, which provide for reversal of the flow. The sewage flow is controlled by so-called "cone orifice" valves. These have proved to be very satisfactory in operation with relatively uniform and complete use of the displacement capacities of the six tanks. The sewage flows out of each tank over weirs 16 ft. long, with suspended screens in front.

The settling tanks are built of reinforced concrete in two units with a main expansion joint between. The side walls are 20 in. thick at the bottom and are tied together by cross-walls heavily reinforced. The baffles forming the settling compartment were built up by a cement gun shooting on to expanded metal, with resulting smooth surfaces and satisfactory results. Each baffle is divided into three sections by vertical expansion joints over each end cross-wall.

Sludge is withdrawn through 8-in. cast-iron sludge pipes having 4.5 ft. of head on the center line of the outlets. These outlets are controlled by sluice-gates and discharge into open conduits leading to the sludge beds. Each of the tanks is fully equipped with a pressure water system.

The first construction work included 40 000 sq. ft. of sludge bed area with provision for an extension (since made) of 16 500 sq. ft. There were originally sixteen sludge beds equally divided on the two sides of a central sludge conduit. Each bed is 100 ft. long by 25 ft. wide, separated from the others by concrete plank partition 2 in. thick. Industrial track for sludge cars are placed along the center of each bed.

Two dosing tanks form part of the operating unit of 3 acres of sprinkling filters. Each tank contains one 30-in. Miller siphon discharging through an independent pipe line to 1.5 acres of filters. The tanks as originally built each held 15 500 gal. of sewage and discharged once every 7 to 9 min. Sloping bottoms were built into the tanks after tests of the operation of the filter distribution system. The maximum rate of flow for maintaining intermittent operation is 7 440 000 gal. per 24 hours.

The sprinkling filters are built in units of 1.5 acres, each 473.5 ft. long and 140.5 ft. wide, with one unit on each side of a common pipe gallery. The depth of filter stone over the under-drains varies from 5.67 to 6.33 ft. The surface of each filter slopes away from the dosing tanks 0.8 ft. in 470. The under-drains are of the trough type covered with vitrified clay blocks and slope to a main collector in the pipe gallery at the rate of 0.67 ft. in 145. Except for a thin layer of larger stones over the under-drains, the filter stone is 1 to 2 in. in size. Each filter unit has 430 round-spray sprinkler nozzles of the Taylor type. The lateral distributors of 6 and 8-in. pipe are spaced 11.5 ft. on centers and the nozzles are 13.25 ft. apart on each lateral. The filter floor is of unreinforced concrete 5 in. thick, built in strips 10 ft. wide.

The effluent from the sprinkling filters flows through a main outfall conduit to the final settling tank. The first construction included only one final settling tank out of four proposed for the ultimate capacity. This tank

is circular and has a liquid depth of 11.19 ft. at the center. The sewage is discharged at the center, 9.11 ft. above the bottom of the tank, and flows radially to weirs at the circumference. The sludge is removed by a Dorr mechanism to sludge pumps of the Barnes type set in an adjacent building. These pumps have a suction lift of about 11 ft., a capacity of 59 gal. per min., and operate satisfactorily. The settling tank is built of reinforced concrete with a bottom 48 in. thick to compensate upward pressure.

At a sewage flow of 8 000 000 gal. per day, the rating of the final settling tank is 1 810 gal. per sq. ft. of tank surface per 24 hours, or a displacement period of 37 min. Sludge is pumped through a wood stave force main over the dike to bottom-land along the river.

The office and laboratory building is about 30 ft. square and contains two floors and a basement. The upper floor is intended for living quarters for workmen, but has never been furnished for use. The basement contains the heating plant, a shop, and space for the sludge locomotive. The intermediate floor is partitioned into four rooms, two of which are fully equipped laboratories, one is the office, and the fourth, a drafting-room. The space appears to be ample. The building is of common brick with asphalt shingles.

The water supply for the plant comes from an infiltration gallery approximately 6 ft. in diameter and delivering by gravity through a 4-in. supply main about 300 ft. long to a centrifugal pump in the basement of the office building. The water is pumped into an elevated tank of 50 000 gal. capacity and is distributed about the grounds.

Trains of six sludge cars are hauled by a Plymouth gasoline locomotive to a sludge dump along the west dike. Some sludge has been pumped wet to adjacent cornfields.

The grounds about the plant are under-drained, but very little landscaping has been undertaken as yet. There is a by-pass sewer for the entire plant and one for the sprinkling filters.

## Operation of Sewage Treatment Works, 1924-1927.

*Quality of Sewage.*—The quantities and characteristics of Decatur sewage are shown in Table 12 which gives the average monthly data. The chemical data are determined according to the Standard Methods of Water Analysis (American Public Health Association, 1923) on daily samples composited hourly and kept at 10 to 20° cent. in a water bath. The data are discussed under three divisions, A, B, and C, as follows:

Division A is the data from August, 1924, through June, 1925. During this period 6 000 000 to 8 000 000 gal. of relatively pure condenser water from the glucose refinery were discharging into the City sewers, thereby increasing the total dry-weather volume of sewage to 15 000 000 gal. per day, or more. Of this quantity, 6 500 000 gal. per day was by-passed around the plant, 8 500 000 gal. per day was passed through the Imhoff tanks, and all but 3 000 000 to 4 000 000 gal. per day was again by-passed around the filters. The data show the sewage of this period to be one of normal suspended matter, high in total nitrogen, oxygen consumed, and bio-chemical oxygen demand, but only one-half as strong as that received during the period discussed under Division B.

Division B is the data from July, 1925, through August, 1926. During this period the refinery condenser water was not entering the sewers, but was being returned to the impounding reservoir. This period was characterized by a normal rainfall and a steady increase in the corn grind from month to month. However, the efficiency of operation of the starch plant was improved after November, 1925, so that the strength of the sewage at higher corn grinds was less than it had previously been at a much lower grind.

TABLE 12.—Analyses of Crude Sewage—Monthly Averages.

| Month. | Total sewage flow, in million gallons daily. | Suspended matter, in parts per million. | Oxygen consumed, in parts per million. | Total nitrogen, in parts per million. | 5-day biochemical oxygen demand, in parts per million. | Total population equivalent. | Remarks. |
|---|---|---|---|---|---|---|---|
| 1924: | | | | | | | |
| August | (15.0)* | 216 | 117 | 34 | 350 | 256 000 | Division A. 6 000 000 to 8 000 000 gal. per day, condenser water, diluting sewage. |
| September | (15.0) | 246 | 173 | 41 | 415 | 305 000 | |
| October | (15.0) | 212 | 165 | 29 | 345 | 253 000 | |
| November | (15.0) | 218 | 184 | 30 | 370 | 272 000 | |
| December | (15.0) | 197 | 231 | 39 | 485 | 356 000 | |
| 1925: | | | | | | | |
| January | (15.0) | 132 | 224 | 45 | 450 | 330 000 | |
| February | (15.0) | 227 | 228 | 45 | 455 | 334 000 | |
| March | (15.0) | 252 | 194 | 33 | 370 | 272 000 | |
| April | (15.0) | 232 | 171 | 26 | 310 | 228 000 | |
| May | (15.0) | 262 | 206 | 30 | 435 | 320 000 | |
| June | (14.20) | 248 | 193 | 40 | 498 | 366 000 | |
| July | 9.25 | 209 | 149 | 31 | 405 | 204 000 | Division B, condenser water diverted. |
| August | 10.80 | 298 | 162 | 30 | 503 | 216 000 | |
| September | 11.20 | 332 | 236 | 51 | 560 | 257 000 | |
| October | 10.45 | 334 | 352 | 60 | 710 | 365 000 | |
| November | 9.74 | 360 | 421 | 74 | 785 | 374 000 | |
| December | 9.53 | 291 | 380 | 77 | 740 | 346 000 | |
| 1926: | | | | | | | |
| January | 9.88 | 324 | 418 | 69 | 748 | 362 000 | |
| February | 11.38 | 322 | 368 | 56 | 735 | 410 000 | |
| March | 11.80 | 305 | 343 | 52 | 663 | 383 000 | |
| April | (13.50)* | 278 | 299 | 42 | 528 | 349 000 | |
| May | (12.50) | 262 | 244 | 58 | 565 | 346 000 | |
| June | (11.50) | 222 | 172 | 42 | 384 | 216 000 | |
| July | 9.70 | 218 | 206 | 50 | 523 | 249 000 | |
| August | 11.50 | 235 | 199 | 52 | 584 | 309 000 | |
| September | (20.40)* | 232 | 168 | 35 | 325 | 325 000 | Division C, flood-water dilution. |
| October | (15.60) | 214 | 185 | 38 | 425 | 325 000 | |
| November | (12.60) | 285 | 236 | 35 | 520 | 321 000 | |
| December | 11.45 | 290 | 260 | 52 | 570 | 320 000 | |
| 1927: | | | | | | | |
| January | 11.30 | 269 | 265 | 52 | 580 | 321 000 | |
| February | 13.40 | 229 | 240 | 47 | 498 | 327 000 | |
| March | 16.10 | 252 | 214 | 41 | 413 | 326 000 | |

* Calculated total sewage, including by-pass.

Division C is the data for eight months of abnormally high rainfall and river stage, with dilution of the sewage by storm water and infiltration. This period also represents further increases in efficiency at the starch works, as indicated by the decrease in population equivalent. The general trend of the chemical quantities was downward, due partly to dilution by infiltration and storm water, and partly to greater efficiencies at the starch works.

*Operating Details.*—The sewage passes first through a bar screen with openings between the bars of $1\frac{9}{16}$ in. From June 1, 1924, to September 1, 1926, a total of 110 cu. yd. of screenings was removed. This represents 0.0153 cu. yd., or 0.4 cu. ft., of screenings per million gallons of sewage. The screenings are buried daily.

The grit chambers are operated alternately and the velocity of flow is entirely controlled by the orifice weir at the outlet. The character of grit usually obtained is mostly sand, cinders, and fine coal. At certain periods of the year there is a tendency for paunch manure from two packing houses to settle toward the outlet ends of the chambers. This material is obnoxious to handle, but dries rapidly and causes no odor where used as fill.

The total of 541.5 cu. yd. of grit, or 2.16 cu. ft. per million gallons of sewage, has been obtained in 24 months of operation. The grit has been used for filling in low places and for replacing sand in the sludge drying beds. It has not been entirely satisfactory for this latter purpose because of its low density, which allows it to adhere to the dry sludge and be removed too rapidly from the beds. The possibility of using a sand-washing machine for reclaiming the heavier sand for use on the drying beds is being considered.

The Imhoff tanks have operated most satisfactorily considering that they have been run most of the time at 35% above their designed capacity. The resulting larger volume, due to the increased industrial sewage and the lack of sludge bed capacity, has caused the sludge to reach the slots of the flowing-through compartments each winter in January, thus reducing the efficiency of the tanks during January, February, and March. Fig. 9 shows the sewage flow and the percentage-removal of suspended matter in the Imhoff tanks to date (1927). The removal of bio-chemical oxygen demand has been between 20 and 22 per cent. This low result is attained because of the large amount of colloidal material in the starch works sewage.

Sewage was started through the Imhoff tanks on May 15, 1924, and vigorous foaming began on July 24, 1924. The foam rose over the free-boards of the gas vents and covered the flowing-through compartment to a depth of about 1 ft. However, well-designed scum-boards held the foam back so that very little of it escaped in the effluent of the tanks. Stirring and hosing the tanks added to the toubles at least temporarily. After about six weeks, the foaming stopped and was not again experienced until after the three-month shut-down in the summer of 1926 for the installation of the gas collectors. During the shut-down the sludge remaining in the tanks practically stopped gassing and apparently did not properly seed the fresh sludge when the tanks were again started in July. Numerous laboratory tests have shown that old sludge may become dead, so to speak, in so far as its inoculating capacity is concerned. Beginning in the fall of 1926, some of the tanks foamed mildly for three months, but in December the foaming became so violent that many of the gas-collecting domes were disconnected to prevent foam from entering the gas-piping system. It became necessary to remove the wooden sludge screen in each gas collector, because the "sludge-scum" which accompanied the foaming was very slimy and clogged the openings in the screen, thus creating considerable pressure. On releasing the screens some were blown from 4 to 6 ft. into the air, and the workmen got a foam shower bath.

Analyses of the gas during foaming show a high carbon dioxide content. The pH of the sludge liquor, at this time, was about 6.4, thus indicating that

the acidity needed correction. To obtain some practical information on these foaming troubles several experiments were made.

One tank was not treated in any way; that is, foaming was allowed to take its own course. This tank foamed continuously for two months.

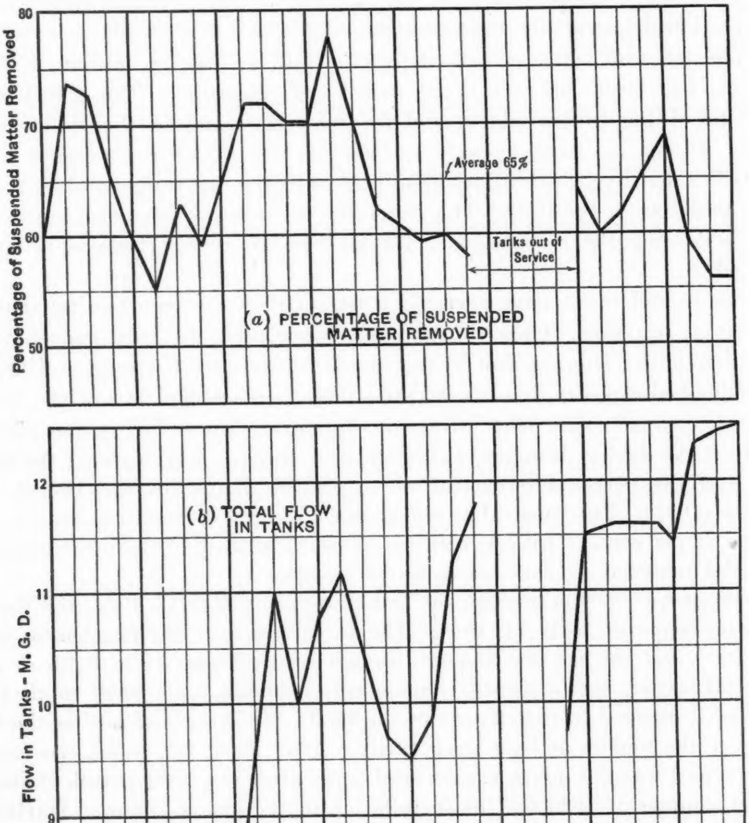


FIG. 9.—RESULTS OF OPERATION OF IMHOFF TANKS.

In one tank the depth of sludge liquor was measured from the surface to the solid sludge layer in the bottom. The acidity of the sludge liquor was titrated with Brom-thymol blue to a pH of 7.0, and the necessary quantity of lime calculated. The lime was added to each hopper down the gas-collector chimneys, through a chute, with a stream of water. This chute delivered the

lime-water mixture about 4 ft. below the surface of the scum. Within 24 hours foaming had stopped, and the gas-collecting system was reconnected and placed in operation.

In another tank the lime was placed on top of the "foam-scum" in the flowing-through compartment. The influent of the tank was shut off and the scum hosed down. Foaming was greatly reduced, although not so effectively as where the lime was introduced directly into the sludge compartment.

In other tanks small quantities of lime were added to the gas vents every few days over a period of 2 to 4 weeks. More lime, more trouble, and more time was necessary to accomplish results in this way than by adding the lime carefully and slowly, but all in one day, as described previously.

Foaming that is not vigorous can be greatly reduced by removing the sludge scum from the gas vents and throwing it into the flowing-through compartments, or, better, by putting it on the drying beds. Regular stirring with water pressure is also helpful in the prevention of foaming.

The foregoing data are based on one period of experience with foaming. The same experience might not occur again, and several years of operation would be required to permit general conclusions.

No record has been kept of the volume of wet digested sludge removed from the Imhoff tanks, but 7 493 cu. yd. of air-dried sludge have been removed from the sludge drying beds. This is equivalent to 1.04 cu. yd. per million gallons of sewage treated.

Because of the high temperatures of the sewage due to the starch waste (70° Fahr. in winter to 104° Fahr. in summer), sludge digestion is very rapid and satisfactory all year around, even if the normal pH is as low as 6.8. At no time has there been any sludge drawn with an obnoxious odor, even when by mistake one tank was completely emptied of its sludge. Laboratory experiments with Decatur conditions have shown that at least 50% of the daily gas produced is given off from freshly deposited sludge in the first 24 hours. If the gas rate from the tanks was 100 000 cu. ft. per day and the tanks were by-passed for 24 hours, the rate would be reduced to 50 000 cu. ft. per day. This rapidity of decomposition probably explains why no foul-smelling gray sludge has ever been obtained, even by collecting a sample at the sludge line in the digestion chamber. It should be noted, however, that sludge drawn when serious foaming was taking place did not dry as rapidly as properly digested sludge, although there was present the usual "tarry odor".

The method of operating the sludge beds has been first to rake the sand smooth and then to run on from 12 to 15 in. of sludge. This sludge will dry in 10 days of good drying weather, and it is then forked into cars and dumped on low ground as fill. Considerable sludge can be taken from the beds during freezing weather. The cakes of sludge freeze to such a consistency that they are almost as easily forked as summer sludge. The sludge-bed capacity has recently been increased 41%, which will relieve the Imhoff digestion chamber of its heavy winter and spring load.

The sprinkling filter operation has been intermittent because its elevation is such that it is not operated at times of high water in the river, and because

the plant was by-passed for three months in 1926 for the construction of gas collectors and for several months in 1927 for the construction of the pre-aeration plant. Since the filters were placed in operation they have operated about 50% of the time. Table 13 gives data on a typical run from May, 1925, until January, 1926. It shows the effects of the concentration of the sewage on the filter rates, loadings, and efficiencies, particularly when high concentration and cold weather coincide. The sprinkling filter rates ranged from 1 500 000 to 750 000 gal. per day as the bio-chemical oxygen demand of the sewage ranged from 500 to 750 parts per million and the sewage temperature from 90° to 70° Fahr. At these rates the total filtering capacity of the 3 acres was from 2 300 000 to 4 500 000 gal. per day, or about one-third that necessary for complete treatment of the entire sewage. A rate of 1 000 000 gal. per day, with a bio-chemical oxygen demand of the sewage of 500 parts per million is equivalent to a bio-chemical oxygen demand loading of 4 165 lb. per acre per day.

Some trouble with ponding has been experienced. This seems to be due to growths on the surface stones and is broken up by forking, harrowing, and with water pressure from a fire hose.

TABLE 13.—Sprinkling Filter Data.

| Month. | Rate of application, in million gallons per day. | 5-day bio-chemical oxygen demand, influent, in parts per million. | Bio-chemical oxygen demand, loading, in pounds per acre per day. | 5-Day Bio-Chemical Oxygen Demand, in Parts per Million. | | Total nitrogen, raw sewage, in parts per million | Total nitrogen, in pounds per acre per day. | Nitrates and nitrites, effluent, in parts per million. | Percentage of stability of effluent to methylene blue. |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Sprinkling filter effluent. | Secondary tank effluent. | | | | |
| 1925: | | | | | | | | | |
| May | 1.7 | 371 | 5 250 | 25 | .... | 30 | 425 | 11.3 | 90 |
| June | 1.76 | 388 | 5 700 | 24 | .... | 40 | 587 | 7.2 | 90 |
| July | 2.0 | 290 | 4 830 | 26 | .... | 31 | 519 | 4.3 | 90 |
| August | 1.5 | 344 | 4 300 | 32 | 20 | 30 | 375 | 7.4 | 90 |
| September | 1.28 | 405 | 4 320 | 25 | 10 | 51 | 545 | 9.2 | 90 |
| October | 1.21 | 530 | 5 340 | 79 | 50 | 60 | 606 | 5.5 | 60 |
| November | 0.74 | 610 | 3 760 | 130 | 70 | 74 | 457 | 6.5 | 40 |
| December | 0.79 | 595 | 3 920 | 71 | 30 | 77 | 455 | 17.0 | 60 |
| 1926: | | | | | | | | | |
| January | 1.06 | 650 | 5 740 | 63 | 30 | 69 | 611 | 20.0 | 90 |

The data in Table 13 show the effective removal (by the secondary tank) of bio-chemical oxygen demand from the sprinkling filter effluent. This bio-chemical oxygen demand appears to come largely from materials sloughing off the filter stones. The suspended matter removal is about 66 per cent.

*Odors.*—Very strong decomposition odors were noticeable at the inlet to the screen chamber when the plant was placed in operation. These odors were due to the relatively high temperature, which caused decomposition of the very strong sewage in the lateral and intercepting sewers. A galvanized iron railing around the screen and grit chamber was turned snow white by zinc sulfide frost. During the first few months of operation, a flap-gate was made which held these gases in the interceptor. This greatly reduced the odor during the first two years of operation.

After the diluting condenser water from the glucose plant was taken out of the sewerage system, the odors about the plant were greatly intensified, particularly at all points of turbulent flow. Analyses of the air about the plant for hydrogen sulfide showed the intensity of odor to exist in about the following order: (*a*) Gas from Imhoff tank gas vents; (*b*) above the grit-chamber effluent near the outlet weir; (*c*) gas from the intercepting sewer; (*d*) the overflow by-pass of the Imhoff tanks; (*e*) the dosing tanks; (*f*) the screen chamber; and (*g*) leeward of sprinkling filters during spray. These results showed the intensity of odor, but did not show the quantity. For example, the total hydrogen sulfide liberated from the 3 acres of sprinkling filters must have been very high, but the dilution with air over so great a surface makes the hydrogen sulfide concentration quite low.

The high hydrogen sulfide content of the Imhoff digestion gas was a surprise. Routine analyses of the Imhoff gas over a period of six months revealed that the hydrogen sulfide content is proportional to the temperature of digestion. Fig. 10 gives the data on temperature of digestion, percentage of hydrogen sulfide present, and the computed total pounds of hydrogen sulfide produced per day from the total Imhoff tank digestion. These data show the large amount (100 lb. per day) of hydrogen sulfide liberated at summer temperatures of digestion.

It has only been possible to obtain quantitative data on the Imhoff gas, but after considerable study of conditions a qualitative estimate indicated that, roughly, 25% of the odors came from the grit chamber, 50% from the Imhoff gas vents, conduits, and by-passes, and probably 25% from the dosing tank and sprinkling filters.

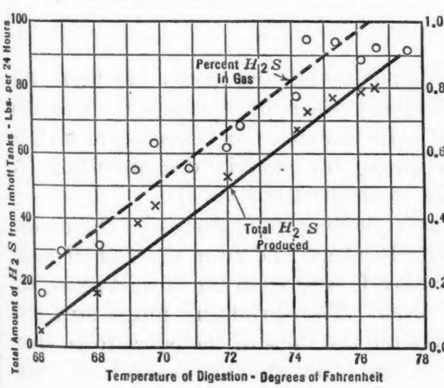


FIG. 10.—HYDROGEN SULFIDE IN IMHOFF TANK GASES, SEWAGE DISPOSAL WORKS, DECATUR, ILL.

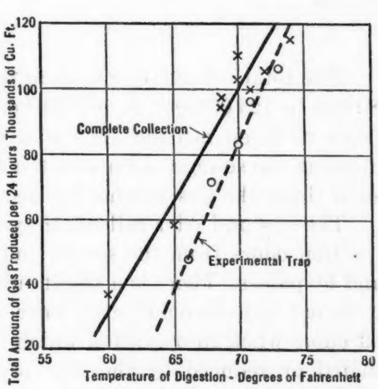


FIG. 11.—VOLUME OF GAS FROM IMHOFF TANKS, SEWAGE DISPOSAL WORKS, DECATUR, ILL.

The odors were reduced by covering the screen chamber, the grit chamber, the Imhoff tank conduits, and by-passes, and collecting all the Imhoff tank digestion gas. The chambers, conduits, and by-passes were covered with a 4 to 6-in. reinforced concrete slab containing suitable manholes and removable covers for accessibility. These covered sections were connected by 8 and 10-in. suction piping to an exhaust fan which forces the gases into a combustion

oven. The details for the collection of digestion gases are given in Fig. 4. The gas is piped to the office building for heating and laboratory use and to the combustion oven for burning, thus destroying the odors of the digestion gas and also the housed-in odors.

*Gas Production.*—Before building the gas collectors the central gas vent of one tank was experimentally trapped by building a sloping wooden structure under the surface of the sewage, somewhat resembling the Imhoff collector.* This structure collected about one-twelfth the entire gas production. The gas from this collector was used in the laboratory and to heat the office and laboratory building during the winters of 1925 and 1926. The data regularly collected on this experimentally trapped gas were temperature of sludge digestion, volume of gas, hydrogen sulfide, calorific value, and chemical constituents. The relations of temperature to hydrogen sulfide and to total volume are shown in Figs. 10 and 11. The gas has an average calorific value of about 700 B.t.u. per cu. ft. The chemical composition varies as shown in Table 14.

TABLE 14.—CHEMICAL COMPOSITION OF GASES.

| Gas. | Percentage by Volume. | | | |
|---|---|---|---|---|
| | Maximum. | Minimum. | Average. | With foaming tanks. |
| Methane | 74 | 55 | 68 | 60–64 |
| Carbon dioxide | 34 | 16 | 22 | 30–33 |
| Hydrogen | 4 | 0 | 2 | 2 |
| Nitrogen | 10 | 4 | 6 | 7 |
| Oxygen | 0 | 0 | 0 | 0 |

The total volume of gas from the complete collecting system has been from 80 000 to 125 000 cu. ft. per 24 hours, except when the starch works was shut down or when melting snow and cold rains lowered the temperature and, by diluting the sewage, decreased the food supply for bacterial decomposition. At such times the gas volume has reduced to about 30 000 cu. ft.

The gas and odor-collecting systems have completely destroyed the decomposition odors from the screen and grit chambers, and from the Imhoff tanks and by-passes. There is a slight sulfur dioxide odor from the burned gases, but it is not objectionable even near the oven. The sprinkling filters still give off odors when in operation which leaves the last 25% of the odors to be eliminated or reduced by the new pre-aeration plant and by recovery of waste products at the starch works. The sprinkling filters were not in operation during the summer of 1927, because of high water and the construction work for the new pre-aeration plant. During this period there have been no odors at all from the other parts of the plant, thus showing its efficiency for collecting and burning the gases.

Utilization of the gas for generation of power for the pre-aeration plant has been considered but, due to the uncertainty of the future volume and strength of the starch works sewage, it did not seem advisable to use it at

* *Engineering News-Record*, Vol. 91, No. 13, p. 512 (1925).

present. From the analyses, about 14 cu. ft. should develop 1 h.p.-hr. in a combustion engine. The gas is to be used for heating the air for the new pre-aeration plant before compression, and is now used for heating the building, for hot water, and in the laboratory.

*Labor Force and Annual Budget.*—The labor force required to operate the plant has been an assistant chemist, a foreman, two full-time operators, a night watchman, a gardener to take care of the grounds and assist in other work, and two extra men during the sludge-drying season. The budget for the fiscal year 1926-27 for operation and maintenance of the plant is as follows:

| Item. | Amount. |
|---|---|
| Supervision | $4 000 |
| Labor payroll | 10 000 |
| Equipment, laboratory and engineering | 600 |
| Materials and supplies | 1 120 |
| Electric power, pumping station and plant | 2 000 |
| Gasoline, oil, and grease | 300 |
| Recording gauge | 150 |
| Landscaping | 1 000 |
| Testing station operation | 2 000 |
| Repairs to roadways | 200 |
| Contingencies | 630 |
| Total | $22 000 |

It is too early to predict the cost of operating the pre-aeration plant. However, it will probably be necessary to add only one more operator to the staff. The power costs for pre-aeration have been estimated at $15 000 per year.

*Laboratory Routine.*—Rather complete analytical data have been kept on the raw sewage and effluents, particularly because of the unusual strength of the sewage and the influence of adjustments by the starch works in its plant process or otherwise.

During 1924 and 1925 the samples were analyzed daily, but beginning in January, 1926, bio-chemical oxygen demand and nitrite-nitrate nitrogen only were determined daily; the other determinations were run on samples made by compositing 2-day samples. This method gave results which are satisfactory and allowed more time for important research work.

The regular routine analyses made were: (*a*) Five-day bio-chemical oxygen demand, daily on raw sewage, experiment station samples, Imhoff effluent, sprinkling filter effluent, and secondary tank effluent; (*b*) suspended matter, settleable matter, permanganate oxygen consumed, total nitrogen, and ammonia were determined on the 2-day composite; and (*c*) nitrite-nitrate nitrogen, methylene blue stability, and dissolved oxygen were determined daily on aeration testing station and sprinkling filter effluents.

All analyses are made according to "Standard Methods of Water Analysis", (American Public Health Association, 1923), except that total nitrogen was determined instead of first boiling off the ammonia and then measuring the organic nitrogen. Ammonia is determined by direct Nesslerization.

The dilution water used for the bio-chemical oxygen demand determination was from the infiltration gallery which supplies the sewage plant water system. Dr. E. R. Greenfield* has described the importance of salts in dilution water and their effect on the 10 and 20-day oxygen demands. Dr. F. W. Mohlman recommends 0.5% $NaHCO_3$ in distilled water. The writers favor the establishment of a definite salt-containing dilution water for the bio-chemical oxygen demand determination. The composition of the tap water which has been used after aeration and standing 20 days is given in Table 15.

TABLE 15.—COMPOSITION OF TAP WATER.
(pH = 8.7).

| Item | Parts per million. | Item | Parts per million. |
|---|---|---|---|
| Calcium (Ca) | 114 | Carbonate ($CO_3$) | 178 |
| Magnesium (Mg) | 57 | Bicarbonate ($HCO_3$) | 0 |
| Sodium and potassium (Na and K) | 12 | Sulfate ($SO_4$) | 112 |
| Iron (Fe) | 0 | Chloride (Cl) | 41 |
| Ammonium ($NH_4$) | 0.01 | Nitrate ($NO_3$) | 15 |

Oxygen consumed from permanganate bears a very definite relation to bio-chemical oxygen demand for each sampling point, for the condition of the sample, and for the period of year. At Decatur the oxygen consumed is used as a convenient yardstick from which is predicted the probable 5-day demand on the first day.

The present Imhoff installation may be relied on to give a 65 to 75% removal of suspended matter and a 22% removal of bio-chemical oxygen demand when treating as much as 11 000 000 gal. per day. This is the maximum volume which it will handle satisfactorily.

The additional sludge bed area, making a total of 1.37 acres (equivalent to 1.0 sq. ft. per capita based on 60 000 population), will give ample sludge capacity in the Imhoff digestion chamber for the sludge from the mixed sewage. There may not be sufficient sludge capacity to take care of the waste activated sludge from the pre-aeration plant which is to be returned for digestion into the Imhoff tanks, owing largely to the uncertainty as to its volume and temperature. Separate sludge digestion tanks may have to be added later.

The sprinkling filter capacities have varied from 700 000 to 2 000 000 gal. per acre per day and from about 4 000 to 5 700 lb. of 5-day bio-chemical oxygen demand per acre per 24 hours. For an average applied sewage strength of 500 bio-chemical oxygen demand and moderate summer temperatures, the sprinkling filter will handle about 1 200 000 gal. per day and 5 000 lb. of 5-day oxygen demand per acre 6 ft. deep. During cold weather the filter rate must be cut to somewhat more than 700 000 gal. per day, in order to keep the filter in condition. The maximum nitrogen loading with Decatur sewage

* *Industrial Engineering Chemistry*, No. 18, p. 1276 (1926).

seems to be about 552 lb. per acre 6 ft. deep. The 3 acres of sprinkling filters have only one-third to one-fourth the capacity required to treat the total mixed sewage without pre-aeration.

### PRE-AERATION TESTING STATION, 1925-1926

*Purpose.*—The purpose of the pre-aeration testing station was to determine the applicability of a combination of aeration and sprinkling filters to sewage treatment at Decatur. Experience with the operation of the main sewage treatment plant indicated that a relatively large area of sprinkling filters would be needed, amounting to about 9 to 12 acres, depending on the process adjustments at the starch works. Such an increase in filter area was beyond the means of the Sanitary District and was considered hazardous from an odor standpoint.

The installation at Birmingham, England, of sewage treatment works, comprising a short period of aeration of settled sewage, with increased loading of existing sprinkling filters, looked promising. A visit to Birmingham was made, therefore, by Paul Hansen, M. Am. Soc. C. E., in 1924 and, later, by one of the Trustees of the District. The construction of the testing station as described was then recommended and authorized. Actual operation began on February 23, 1925.

*Operating Records.*—A complete record of operation is given in Table 16. The influent to the testing station comprised settled sewage from the Imhoff tanks. The volume of this influent sewage, the sprinkling filter influent, the returned sludge, and the excess sludge wasted to the sewer were all measured in orifice boxes. The volume of air used for sludge re-aeration was measured by a displacement meter. The power for operating the Simplex aerator was measured by a recording watt-hour meter. The orifice boxes for measuring the influent sewage and the liquor going to the sprinkling filters were provided with constant level overflows so that the flow through the orifice was constant. The orifice boxes for the sludge did not have constant level overflows. However, by careful attention, a close record was kept of the sludge quantities despite the difficulties occasioned by the wide variation in the density of the sludge.

The sprinkling filter was first dosed from a tank the outlet of which was controlled by a cam which opened and closed a valve in the influent line to the filter. The operation of this method of control was satisfactory, but required numerous cams for changing the rate of flow and very careful adjustment of the sprinkling filter orifice box in order not to overflow the dosing tank. The cam, therefore, was displaced by a siphon which delivered sewage to the filter at the rate at which it was pumped to the siphon tank. The air vents in so small a siphon were occasionally plugged with slimy growths so that they had to be cleaned regularly. The sprinkling filter was made of stone from the large filters and, therefore, was seeded from the beginning of the experiments.

*Operating Periods.*—The testing station was operated practically continuously from February 23 to August 20, 1925, when the motor for the Simplex aerator burned out, occasioning a shut-down until September 11. During

TABLE 16.—RECORDS OF OPERATION OF TESTING STATION.

| Period. | Month and year. | Percentage of sludge return. | Percentage of sludge wasted to sewer. | Simplex aeration, in hours. | Re-aeration of sludge, in hours. | Cubic feet of air per gallon of sewage. | Displacement period, in hours, settling tank. | Filter rate, in million gallons per day. | Bio-Chemical Oxygen Demand,* 5 Days, 20° Cent. | | | | Percentage of Removal, Bio-Chemical Oxygen Demand.† | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | Crude sewage. | Imhoff tank effluent. | Aeration tank effluent. | Sprinkling filter effluent. | Imhoff tanks. | Aeration. | Filter. | Total |
| 1 | 2/25 | 6.0 | 0 | 5.2 | 3.2 | 3.4 | 1.0 | 1.24 | (397) | (318) | (190) | (52) | .... | 40 | .... | .... |
| 2 | 3/25 | 25.0 | 12 | 4.3 | 2.8 | 2.1 | 1.1 | 2.51 | (305) | (244) | (104) | (24) | .... | 57 | .... | .... |
| 3 | 4/25 | 22.2 | 0 | 7.2 | 5.8 | 5.6 | 1.6 | 3.00 | (303) | (243) | (152) | (28) | 20 | 38 | .... | 92 |
| 4 | 4/25 | 16.3 | 1 | 3.5 | 3.1 | 3.26 | 0.8 | 3.05 | (352) | (282) | (148) | (28) | .... | 48 | .... | .... |
| 5 | 5/25 | 36.0 | 15.8 | 3.1 | 1.7 | 1.94 | .... | 3.05 | (450) | (360) | (180) | (28) | .... | 64 | .... | .... |
| 6 | 5/25 | 21.0 | 18.7 | 3.4 | .... | .... | 2.1 | 3.00 | 457 | 371 | 120 | 37 | 19 | 65 | 71 | 92 |
| 7 | 6/25 | 34.0 | 25.0 | 4.1 | .... | .... | Varying | 3.10 | 487 | 408 | 185 | 48 | 16 | 55 | 74 | 90 |
| 8 | 6/25 | 25.0 | 20.8 | 4.5 | .... | .... | 3.0 | 3.10 | 488 | 366 | 128 | 58 | 15 | 65 | 55 | 88 |
| 9 | 7/25 | 34.0 | 5.2 | 6.0 | .... | .... | 3.7 | 3.20 | 374 | 256 | 103 | 25 | 31 | 60 | 76 | 93 |
| 10 | 8/25 | 10.0 | 14.8 | 7.0 | .... | .... | 4.1 | 3.30 | 495 | 340 | 122 | 37 | 31 | 64 | 70 | 93 |
| 11 | 9/25 | 25.0 | (?) | 3.8 | .... | .... | 1.8 | 3.5‡ | 452 | 328 | 146 | 31 | 28 | 56 | 79 | 93 |
| 12 | 9/25 | 13.7 | 0.3 | 3.8 | .... | .... | 1.6 | 3.78 | 543 | 405 | 195 | 44 | 26 | 52 | 77 | 92 |
| 13 | 10/25 | 13.7 | 1.9 | 3.8 | 3.4 | 2.3 | 1.6 | 3.5§ | 761 | 551 | 237 | 38 | 28 | 58 | 84 | 95 |
| 14 | 10/25 | 13.7 | (?)0.9 | 3.8 | .... | .... | 1.6 | 3.36 | 745 | 560 | 269 | 52 | 25 | 52 | 81 | 93 |
| 15 | 11/25 | 10.0 | 1.1 | 2.5 | .... | .... | 1.1 | 3.36 | 787 | 640 | 387 | 67 | 19 | 39 | 83 | 91 |
| 16 | 12/25 | 10.0 | 0.9 | 2.5 | .... | .... | 1.1 | 1.75 | 790 | 608 | 317 | 69 | 23 | 48 | 78 | 91 |
| 17 | 12/25 | 10.0 | 1.0 | 2.5 | .... | .... | 1.1 | 2.52 | 785 | 633 | 430 | 72 | 19 | 32 | 83 | 91 |
| 18 | 1/26 | 11.0 | 0.9 | 3.4 | .... | .... | 1.6 | 2.52 | 760 | 645 | 425 | 103 | 26 | 34 | 76 | 86 |
| 19 | 1/26 | 13.5 | (?) | 3.4 | .... | .... | 1.6 | 2.37 | 770 | 607 | 340 | 58 | 21 | 44 | 83 | 92 |
| 20 | 2/26 | 11.0 | 1.9 | 3.4 | .... | .... | 1.6 | 3.00 | 783 | 606 | 284 | 54 | 23 | 53 | 81 | 93 |
| 21 | 2/26 | 13.0 | 2.4 | 6.7 | .... | .... | 3.3 | {3.00, 3.33, 4.32, 5.00} | 640 | 570 | 118 | 41 | 11 | 79 | 65 | 94 |
| 22 | 3/26 | 13.0 | 2.5 | 9.3 | .... | .... | {4.5, 2.2} | 4.28 | 640 | 535 | 110 | 39 | 16 | 83 | 64 | 94 |
| 23 | 4/26 | 16.0 | 4.8 | 11.2 | .... | ..... | 3.1 | 3.3 | 536 | 491 | 46 | 16 | 11 | 91 | 65 | 97 |

* Bio-chemical oxygen demand data in parentheses are calculated by factors from oxygen consumed and total nitrogen.

† Percentage removal, bio-chemical oxygen demand, is calculated on each step in the process separately and the last column contains the total percentage removal.

‡ Filter rate 1.64, two days; 2.49, two days; 4.21, eight days; average, 3.5.

§ Filter rate 3.78, eight days; 3.33, twelve days.

TABLE 16.—(*Continued*).

| Final Effluent. Relative Stability, Percentage. | Final Effluent. Nitrate and Nitrite Nitrogen. | | Total Organic and Free Nitrogen, $NH_3$, by Kjeldahl. | | | | Oxygen Consumed, $KMnO_4$, 30 Min. | | | | Suspended Matter, Gooch. | | | | Settleable Matter. | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Sprinkling filter effluent. | Aeration tank effluent. | Sprinkling filter effluent. | Crude sewage. | Imhoff tank effluent. | Aeration tank effluent. | Sprinkling filter effluent. | Crude sewage. | Imhoff tank effluent. | Aeration tank effluent. | Sprinkling filter effluent. | Crude sewage. | Imhoff tank effluent. | Aeration tank effluent. | Sprinkling filter effluent. | Crude sewage | Imhoff tank effluent. | Aeration tank effluent. | Sprinkling filter effluent. |
| 69 | .... | 1.6 | 47 | 37 | 34 | .... | 277 | 153 | 95 | 53 | 216 | 92 | 79 | 55 | 2.2 | 0.4 | 0.3 | 0.8 |
| 90+ | 1.6 | 8.1 | 32 | 28 | 26 | .... | 151 | 133 | 52 | 35 | 270 | 100 | 123 | 142 | 2.7 | 0.6 | 13.6 | 6.3 |
| 90+ | 0.8 | 6.7 | 28 | 28 | 29 | .... | 150 | 111 | 76 | 44 | 215 | 77 | 197 | 130 | 3.6 | 0.4 | 16.9 | 2.7 |
| 90+ | 0.0 | 8.7 | 26 | 25 | 23 | .... | 174 | 114 | 74 | 44 | 248 | 76 | 164 | 105 | 4.0 | 0.7 | 39.0 | 12.0 |
| 90+ | 0.1 | 6.8 | 32 | 28 | 21 | .... | 210 | 138 | 65 | 43 | 269 | 75 | 131 | 93 | 4.2 | 0.7 | 93.0 | 37.0 |
| 70 | 0.4 | 5.0 | 32 | 28 | 26 | 9 | 210 | 108 | 85 | 38 | 251 | 78 | 48 | 82 | 4.0 | 0.6 | 77.7 | 2.8 |
| 80+ | 0.4 | 7.2 | 34 | 22 | 22 | 10 | 205 | 119 | 81 | 44 | 266 | 68 | 83 | 114 | 4.7 | 0.2 | 16.7 | 5.5 |
| 80+ | 0.1 | 5.8 | 42 | 38 | 28 | 15 | 191 | 102 | 83 | 56 | 244 | 73 | 107 | 99 | 4.0 | 0.5 | 39.0 | 14.5 |
| 90+ | 0.1 | 6.3 | 30 | 25 | 17 | 6 | 140 | 69 | 44 | 25 | 197 | 61 | 36 | 52 | 3.3 | 0.6 | 90.0 | 7.6 |
| 38 | 0.1 | 9.0 | 32 | .... | .... | .... | 154 | 87 | 63 | 35 | 280 | 89 | .... | .... | 3.7 | 0.9 | 58.0 | 17.3 |
| 53+ | 0.3 | 5.9 | 45 | .... | 38 | 22 | 199 | 125 | 67 | 39 | 322 | 61 | 50 | 28 | 5.8 | 0.8 | 0.3 | 0.5 |
| 85+ | 0.1 | 11.0 | 55 | 58 | 42 | .... | 231 | 133 | 80 | 39 | 293 | 79 | 45 | 56 | 4.7 | 1.2 | 0.3 | 2.3 |
| 79+ | 0.2 | 7.6 | 60 | 60 | 43 | .... | 375 | 196 | 96 | 36 | 354 | 95 | 88 | 36 | 4.6 | 0.9 | 0.2 | 1.2 |
| 90 | 1.3 | 8.9 | 69 | 63 | 49 | .... | 489 | 249 | 116 | 57 | 367 | 127 | 99 | 86 | 4.8 | .... | 0.1 | 3.3 |
| 54 | 2.0 | 6.6 | 79 | 76 | 62 | .... | 396 | 254 | 125 | 54 | 356 | 124 | 77 | 59 | 5.0 | .... | .... | 1.8 |
| 49+ | 1.0 | 14.0 | 72 | 69 | 59 | .... | 382 | 257 | 106 | 62 | 294 | 118 | 73 | 101 | 4.1 | 0.9 | 0.2 | 6.3 |
| 48 | 1.8 | 3.3 | 82 | 77 | 62 | .... | 398 | 264 | 112 | 63 | 315 | 115 | 57 | 70 | 4.4 | 0.6 | 0.2 | 2.1 |
| 11 | .... | 3.3 | 68 | 67 | 59 | .... | 415 | 270 | 120 | 74 | 306 | 124 | 61 | 67 | 4.3 | 0.6 | 0.1 | 2.2 |
| 87 | .... | 12.0 | 69 | 65 | 52 | .... | 393 | 251 | 112 | 60 | 377 | 154 | 42 | 94 | 5.3 | 0.6 | 0.1 | 3.7 |
| 77 | .... | 10.6 | 51 | 48 | 41 | .... | 403 | 265 | 103 | 58 | 350 | 140 | 46 | 78 | 4.0 | 0.7 | 0.1 | 3.1 |
| 99 | 3.5 | 17.8<br>25.0<br>17.0<br>13.0 | 50 | 48 | 35 | .... | 354 | 254 | 79 | 45 | 282 | 122 | 64 | 47 | 3.7 | 0.8 | 0.1 | 1.6 |
| 99 | 2.5 | 19.6 | 33 | 50 | 37 | .... | 319 | 260 | 73 | 52 | 310 | 141 | 84 | 40 | 4.5 | 0.9 | 10.4 | 1.6 |
| 99 | 2.7 | 18.0 | 43 | 51 | 16 | .... | 328 | 263 | 71 | 40 | 289 | 147 | 49 | 23 | 2.9 | 1.0 | 2.7 | 0.4 |

this time the operation was divided into ten periods, each period comprising a run of 12 to 25 days. During each of these periods the various operating elements, such as the rate of flow, the displacement period in the aeration tank, the quantity of sludge returned, and the rate on the sprinkling filters were kept constant. During the first nine periods the sewage contained the diluting refinery condenser water which caused a weaker mixed sewage than was received later during the tests. The sewage during the tenth period was increased in strength by the removal of the condenser water and decreased by a relatively low corn grind. During these first six months of operation much was learned about the behavior of an under-aerated sewage and sludge, and this period may be designated as experimental.

Operation was resumed on September 11, 1925, and continued without any appreciable interruption until April 7, 1926. During this time there were thirteen additional operating periods each of 12 to 20 days' duration, during which the rates of flow, the aeration periods, the quantity of sludge returned, and the sprinkling filter rates were held constant; except that during Runs 11, 13, and 21, the sprinkling filter rates were varied as indicated in Table 17. The crude sewage during this period was typical of the normal strong mixed sewage without dilution by the refinery condenser water or unusual rainfall. Its strength was influenced by considerable fluctuations in the corn grind. The strongest sewage was received during the coldest weather when rates of oxidation were at a minimum and difficulties with frozen pipes somewhat interfered with continuous operation.

TABLE 17.—SPRINKLING FILTER LOADINGS.

| Period No. | Bio-chemical oxygen demand, parts per million, raw sewage. | Hours, aeration. | Bio-chemical oxygen demand, parts per million, influent. | Filter rate, in million gallons per day. | Bio-chemical oxygen demand, pounds per acre per day. | DATA ON SPRINKLING FILTER EFFLUENT. | | | TOTAL NITROGEN. | | POUNDS TOTAL NITROGEN PER ACRE PER DAY. | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Bio-chemical oxygen demand, parts per million. | Nitrate, parts per million. | Stability, percentage. | Raw sewage. | Filter influent. | Raw sewage. | Filter influent. |
| 11 | 452′ | 3.3 | 146 | 3.5* | 4 250 | 31 | 5.9 | 53.9— | 45 | 38 | 1 320 | 1 110 |
| 12 | 543 | 3.3 | 195 | 3.78 | 6 150 | 44 | 11.0 | 85 — | 55 | 42 | 1 730 | 1 330 |
| 13 | 761 | 3.3 | 237 | 3.5* | 6 900 | 38 | 7.6 | 79 — | 60 | 43 | 1 750 | 1 260 |
| 14 | 745 | 3.3 | 269 | 3.36 | 7 550 | 52 | 8.9 | 90 | 69 | 49 | 1 950 | 1 370 |
| 15 | 787 | 2.5 | 387 | 3.36 | 10 800 | 67 | 6 6 | 54 | 79 | 62 | 2 210 | 1 740 |
| 16 | 790 | 2.5 | 317 | 1.75 | 4 620 | 69 | 14.0 | 89 — | 72 | 59 | 1 050 | 860 |
| 17 | 785 | 2.5 | 430 | 2.52 | 9 030 | 72 | 3.3 | 33 | 82 | 62 | 1 730 | 1 310 |
| 18 | 760 | 3.4 | 425 | 2.52 | 8 920 | 103 | 3.3 | 11 | 68 | 59 | 1 430 | 1 240 |
| 19 | 770 | 3.4 | 340 | 2.37 | 6 720 | 58 | 12.0 | 87 | 69 | 52 | 1 360 | 1 030 |
| 20 | 783 | 3.4 | 284 | 3.00 | 7 100 | 54 | 10.6 | 77 | 51 | 41 | 1 280 | 1 030 |
| 21 | 640 | 6.7 | 118 | 3—5 | 2950-4910 | 41 | 18.-13. | 99 | 50 | 35 | 1250-2080 | 880-1460 |
| 22 | 640 | 9.3 | 110 | 4.28 | 3 920 | 39 | 19 6 | 99 | 53 | 37 | 1 900 | 1 320 |
| 23 | 536 | 11.2 | 46 | 3.3 | 1 260 | 16 | 18.0 | 99 | 43 | 16 | 1 190 | 440 |

* Average.

*Pre-Aeration Sludge.*—One of the major problems in the operation of the testing station was to keep the under-aërated sludge in a settleable condition.

The data in Table 16 giving settleable matter as cubic centimeters per liter, show that during the first ten periods of operation a considerable volume of sludge was discharged over the settling-tank weirs on to the sprinkling filter. The relatively light character of this settleable matter is indicated by the proportionately lower quantities of suspended matter. During these periods the sludge was very thin and often would not settle appreciably in 2 hours. This thin fluffy condition of the sludge was first attributed to a lack of sufficiently heavy solids in the settled influent to the aeration tank, and attempts were made to procure a thicker sludge by (*a*) returning a larger percentage of sludge to the aeration tank; (*b*) increasing the period of sludge re-aeration to 5 hours; and (*c*), increasing the aeration period to 7 hours. None of these methods was entirely satisfactory. Increasing the quantity of returned sludge made bulking worse; and increasing the time of sewage aeration and sludge re-aeration, within the limits used, had no appreciable effect. It was finally found that each period of aeration and its strength of sewage gave a characteristic sludge which could be maintained and settled if not more than about 10% by volume of sludge after settling 1 hour was kept in the aeration tank liquor.

Operating experience clearly indicated that sludge should not be allowed to remain or accumulate in the settling tank. The solution of the sludge-handling problem appears to be a prompt and regular drawing of sludge so as to maintain only relatively fresh sludge in circulation as shown during Periods 17, 18, 19, and 20, in Table 16. During Periods 17, 18, and 19, the sludge was black, flocculant, with a rather septic odor, and settled rapidly. During Period 19 a considerable quantity of sludge was lost because of frozen sludge pipes. Just before the start of Period 20 the sludge line broke and all sludge was lost so that during this period the operation of the aeration tank was with entirely fresh sludge. The removal of 5-day bio-chemical oxygen demand by the partial aeration during these periods was, as follows:

| Period. | Percentage removal of 5-day bio-chemical oxygen demand. |
|---|---|
| 17 | 32 |
| 18 | 34 |
| 19 | 44 |
| 20 | 53 |

Thus, with the fresher sludges the removal of bio-chemical oxygen demand was greater.

*Settling Tanks.*—With partial aeration, the condition of the sludge makes the capacity and rating of the settling tanks important. In the testing station, the sludge was satisfactorily settled with a displacement period of 1.1 hours and a surface rating of 1 050 gal. per sq. ft. per 24 hours. When severe bulking of the sludge took place, it could not be settled with the maximum available displacement period of 3.6 hours equivalent to a surface rating of 320 gal. per sq. ft. per 24 hours. With regular and sufficient removal of the excess, the sludge may be kept sufficiently fresh and active and in condition to permit satisfactory sedimentation. This active sludge is in no way a typical activated sludge from complete aeration works, except in the

physical nature of the floc. The color is gray to black and not brown. The odor is disagreeable, but not septic. The operation of the testing station indicated that the volume of sludge in the aeration tanks should not be more than 10% by volume as determined by sedimentation in cylinders for 1 hour. If a poorly settled sludge is developed, the best procedure is to remove it and to start again building up a new sludge. This is often almost white, but becomes more gray in color each day.

*Periods of Aeration.*—The period of aeration has a very definite relation to the amount of purification as measured by the percentage removal of bio-chemical oxygen demand. Fig. 12 shows the reduction in the 5-day bio-chemical oxygen demand for different periods of aeration. Although the longest aeration period was 11.2 hours, the trend of the operating data indicated that about 16 hours might be required for complete treatment of the Decatur sewage after preliminary sedimentation in order to turn out an effluent containing sufficient nitrates to insure stability with maintenance of a satisfactory sludge.

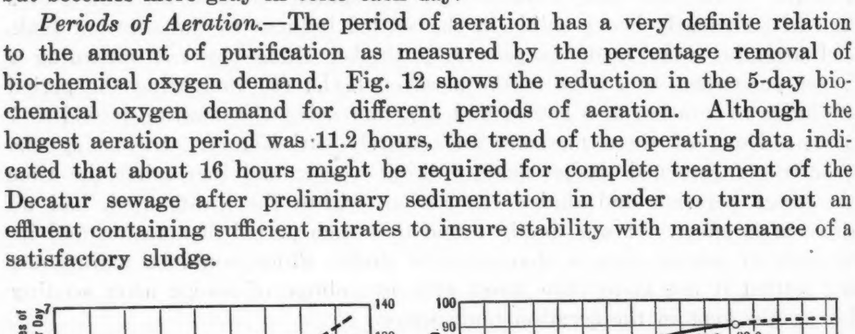


FIG. 12.—PRE-AERATION REDUCTION IN BIO-CHEMICAL OXYGEN DEMAND, SEWAGE DISPOSAL WORKS, DECATUR, ILL.

The maximum capacity of the testing station permitted only 6 hours re-aeration of the returned sludge. Under Decatur conditions this re-aeration did not materially improve the quality of the sludge. It was considered that a larger capacity in the pre-aeration tanks was preferable to provision for sludge re-aeration. In the full-sized plant connections have been made for the later construction of sludge re-aeration tanks should they be found desirable. The testing station operation showed that 6 hours of sludge re-aeration are of little value.

*Operation of Sprinkling Filter.*—The results of operating the small sprinkling filter are shown in Table 17. These data indicate that on a filter 6 ft. deep, about 7 500 lb. of 5-day bio-chemical oxygen demand can be satisfactorily handled per acre per 24 hours. It may be possible to handle a larger quantity than this at summer temperatures. This loading, under Decatur conditions, represents a dose at the rate of between 2 500 000 and 3 500 000 gal. per acre per 24 hours.

Table 16 gives the removal of 5-day bio-chemical oxygen demand by the several parts of the plant. From crude sewage to sprinkling filter effluent the removal of 5-day bio-chemical oxygen demand was, in general, somewhat

more than 90 per cent. The testing station included no secondary or final settling tank for the sprinkling filter effluent. By such sedimentation, the bio-chemical demand of the sprinkling filter effluent may be further reduced from 20 to 50% of the final settling tank influent, thus increasing the over-all removal of bio-chemical oxygen demand to about 95 per cent.

Studies at the testing station showed that within reasonable limits, the sprinkling filters could be operated at a relatively constant loading of 5-day bio-chemical oxygen demand (in the partly aerated sewage) amounting to 7 500 lb. per acre per 24 hours. Thus, the quantity of effluent delivered to the sprinkling filter was proportionate to its strength and was fixed at about 22 500 lb. of bio-chemical oxygen demand for the 3 acres. It is possible, therefore, to draw a rough balance between the amount of pre-aeration necessary for any given strength of sewage with the 3 acres of sprinkling filters. Such a computation would include the over-all annual costs reflecting the fixed charge for construction and the annual cost of power and labor, so that the best economy for each condition might be secured. Fig. 12 gives the amount of aeration needed for different strengths of sewage calculated for dosing 3 acres of filters at the foregoing loads.

Odors from the aeration tank and sprinkling filter were not noticeable during the first ten operating periods, while the strength of sewage was less than 500 parts per million of 5-day bio-chemical oxygen demand, but a very decided "animal den" odor was observed about the aeration tank as the strength of the sewage increased to more than 600 parts per million of 5-day bio-chemical oxygen demand. This odor may have been due to the stronger sewage, but was more probably occasioned by decreased aeration and a more septic sludge. With the process adjustments at the starch works the mixed sewage to be handled in the larger plant will have a 5-day bio-chemical oxygen demand of about 300 parts per million. With this weaker sewage, the new plant is not expected to give off any objectionable odors.

*Basis of Design, New Pre-Aeration Plant.*—The design of the new pre-aeration plant is based on a dry-weather sewage flow of 10 000 000 gal. per day and a maximum rate of 16 000 000 gal. per day, with additional capacity in the aeration tanks for 10% of the returned sludge. Analyses of the starch works sewage and the city sewage indicate that after the process adjustments at the starch works the mixed sewage will have a 5-day bio-chemical oxygen demand of somewhat more than 300 parts per million. Fig. 12 indicates that a sewage of this strength will require pre-aeration for about 1.25 hours to prepare it for application to the existing 3 acres of sprinkling filters. Because of possible irregularities in the starch works sewage and because of the retarding effect of cold and wet weather, it was decided to give the aeration tanks a displacement period exclusive of returned sludge of 2.75 hours, when the sewage flow was 10 000 000 gal. per day. This is equivalent to 1.56 hours at the higher rate of 16 000 000 gal. per day.

The settling tanks were also designed on a relatively conservative basis so that occasional sludge, difficult to settle, can be better recovered. Therefore, a displacement period of 2.6 hours, equivalent to a surface rating of 833 gal. per sq. ft. per 24 hours, was adopted. These capacities are about twice those found desirable during the test periods, 11 to 20 (Table 17).

## Pre-Aeration Plant, 1926-1927

*General Arrangement.*—After the testing station had been put in operation plans were prepared and contracts let for the construction of a pre-aeration plant with a rated capacity of 10 000 000 gal. per day. The plant comprises aeration units, settling tanks, blower house, and appurtenances; and brings the capacity of the sewage disposal works to about 150 000 equivalent population.

During the operation of the pre-aeration testing station, conferences were held with the officials of the starch works relative to process adjustments within the starch plant looking toward a reduction in the strength of the sewage. At that time the organic matter in the starch works sewage was roughly indicated, as follows:

| Plant. | Pounds of total organic and free ammonia nitrogen in starch works sewage per bushel of grind. |
|---|---|
| Argo | 0.06 |
| Pekin | 0.02 |
| Decatur | 0.14 |

The officials of the starch works at Decatur agreed to make process adjustments in their plant. At the time of the conferences in 1926, the equivalent population of the starch works sewage had, at times, exceeded 300 000. It was agreed that process adjustments would be made in the starch works so as to reduce the population equivalent to about 90 000 by July 1, 1927; and thereafter to a population equivalent of about 1 000 for each 1 000 bushels of corn ground and at a rate to keep pace more or less with the growth of the city. The Sanitary District agreed to install a pre-aeration plant to bring the capacity of the disposal works up to about 150 000 by July 1, 1927. Both agreements have been fulfilled except as to time, which has been extended about four months. Thus, the Sanitary District in effect provides a somewhat liberal capacity for its future requirements and permits the starch works to utilize the available capacity for the future.

The pre-aeration plant has been built east of the present sprinkling filters, in the west end of the space alloted to future sprinkling filters in the original layout. The new blower house stands between the aeration tanks and the sprinkling filters. Along the east side of the new settling tanks, space is reserved for sludge re-aeration tanks should they be found desirable; and, in general, future extensions of the pre-aeration plant are planned in an easterly direction.

In order to provide head for the new units, it was not considered necessary to raise the elevation of the flow line in the Imhoff tanks, although this was possible because of the contingent allowance of 1.0 ft. in the original design and the relatively ample free-board of 23 in. The operating level of the dosing tanks however, was adjusted from Elevation 601.90 to Elevation 601.25, thus providing a head of 1.36 ft. for the operation of the pre-aeration plant. The hydraulic computations were based on a maximum rate of sewage flow of 16 000 000 gal. per day, an average of 10 000 000 and a minimum of 8 000 000 gal. per day.

*Aeration Units.*—There are six aeration units each 120 ft. long, 16 ft. wide, and 14.5 ft. liquid depth over the aeration plates, with 14 in. of free-board. Including an allowance of 10% for returned sludge, the aeration tanks have a displacement period of 2.5 hours with a sewage flow of 10 000 000 gal. per day.

The aeration provides spiral flow from two rows of filtros plates along one side of each tank (the so-called Manchester type), the plate area amounting to 10.65% of the tank area.

An inlet conduit extends along the north end of the six tanks from which the sewage flows into the center of each tank through 24 by 24-in. sluice-gates. Return sludge is pumped into the sewage at the inlet or west end of this conduit. The aerated sewage flows out of the tanks over weirs and then through Venturi meters to the settling tanks. There is one Venturi meter for each two aeration tanks so that the sewage flow can be distributed as desired. Indication of this flow is carried by air to the sludge pump houses. A pipe gallery extends along the south end of the aeration tanks, with a sludge pump house at the southeast end of the gallery housing the Venturi meter indicators, valves, piping, and sludge pumps, and connecting directly with the blower house at the west end.

There are two settling tanks each 77.5 ft. square, with a liquid depth of 10.37 ft. at the periphery and 13.69 ft. at the center. The total displacement period above the sloping bottom is 2.6 hours and the settling rate is 833 gal. per sq. ft. of tank surface per 24 hours, both for a sewage rate of 10 000 000 gal. per day.

The sewage will enter the tanks from a conduit along their east sides through six inlets for each tank. The effluent is discharged over adjustable weirs extending along the entire width of each tank and into a conduit connected to the dosing tank.

Each tank is equipped with a Dorr mechanism for the removal of sludge. The sludge passes through an 8-in., cast-iron pipe built under the tanks to an open well at the east end of the pipe gallery. Two 8-in. centrifugal pumps send the sludge either to the main conduit leading to the Imhoff tanks or to the conduit leading to the aeration tanks. There is a Venturi meter for each sludge line, and the division of flow is controlled by gate valves in the sludge lines.

The blower house is 50 ft. long by 26.5 ft. wide inside and has space for four blowers, a boiler for heating the air, and other appurtenances. There are two gear-connected General Electric blowers of the centrifugal type, operating at 11 850 rev. per min., each having a capacity of 3 500 cu. ft. of free air per min. At the average sewage flow of 10 000 000 gal. per day, this is an installed capacity equivalent to about 1.0 cu. ft. of air per gal. of sewage.

The characteristic curves of this type of blower are shown in Fig. 13. Curves *A*, *B*, and *C* are based on air temperatures of 35°, 50°, and 95° Fahr., respectively, and barometric pressures of 14.85, 14.4, and 14.0, respectively. With one blower in operation, the rate of air delivery can be varied from about 0.4 to 0.6 cu. ft. per gal. of sewage, with very little loss in efficiency.

The characteristic curves show that for equal volumes of free air, the warmer the air the less the power required. In view of the fact that considerable gas from the Imhoff tank was available, it was considered advisable to maintain the air going to the blowers at a temperature of about 95° Fahr. The air is drawn through louvres in the blower-house wall, through thermostatically controlled steam-heating coils, through a cell-type air filter, and thence to the blowers. The coils have a capacity to raise 3 500 cu. ft. of air per min., about 70 to 100° Fahr., depending on the temperature of the incoming air. The coils are supplied with steam from the blower-house steam-heating system. The cell filters have a capacity of 7 500 cu. ft. per min.

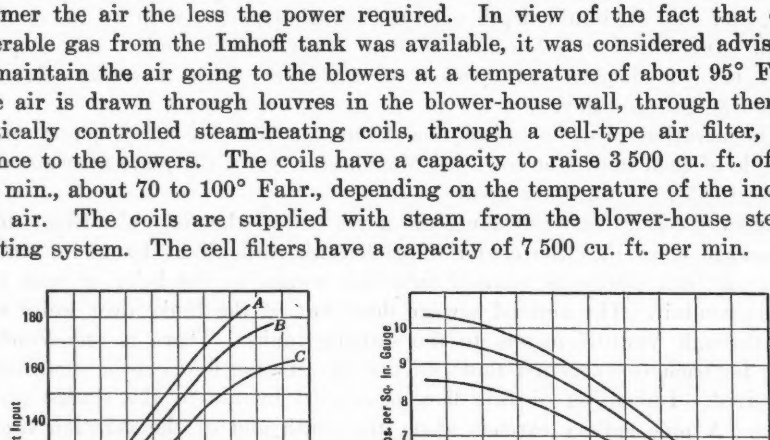


FIG. 13.—CHARACTERISTIC CURVES OF CENTRIFUGAL COMPRESSOR, SEWAGE DISPOSAL WORKS, DECATUR, ILL.

Air deliveries will be measured through a Bailey meter in the blower house on the main 18-in. cast-iron air line. This air line passes through the pipe gallery with 10-in. branches to each aeration tank and a 4-in. branch to all conduits. The rate of air flow to the three 10-in. and one 4-in. air lines is indicated in the sludge pump house by manometers which indicate the differential across the flow nozzles inserted in the several air lines. In this way, the flow in all the branches may be readily distributed by suitable adjustment of the valves. The air lines are provided with condensate traps for the removal of water. The blower house is built of common brick on a concrete foundation and has an asphalt shingle roof.

Excess sludge is pumped into the crude sewage near the bottom of the riser conduit at the northeast corner of the Imhoff tanks. The excess activated sludge is thus being digested with fresh sludge in the lower compartments. With the reduced strength of the starch works sewage, it is expected that there will be sufficient sludge digestion capacity in the present tanks to handle the excess activated sludge. Within a few years, however, it is planned to increase the sludge digestion capacity by adding separate sludge digestion tanks.

The operation of the testing station indicated the possibility of running the sprinkling filters at a rate of 3 000 000 gal. per day per acre. Therefore, it was necessary to increase the capacity of the dosing tanks, to install additional siphons, and to put in many more nozzles.

With the original installation of dosing tanks and 860 nozzles for the 3 acres of filters, the maximum rate of dosing was about 2 500 000 gal. per day

per acre, or a total of 7 500 000 gal. per day, without putting the siphons into continuous operation. The nozzles were originally of the Taylor circular spray type with $\frac{15}{16}$-in. orifices spaced at 13 ft. 3 in., center to center. They were replaced with 1 312 Taylor circular spray nozzles with $\frac{3}{4}$-in. orifices spaced at 8 ft. $10\frac{1}{4}$ in., center to center. The original system of distribution piping was used, and the additional nozzles were installed by means of branch pipes from the taps already made in the lateral distribution. The closer spacing and greater number of nozzles allowed a lowering of the high-water level in the dosing tank of 0.65 ft., which was made available for the pre-aeration plant.

The remodeled dosing tanks and filter distribution system are designed to handle an average flow of 10 000 000 gal. per day, with uniform distribution and can take care of nearly 16 000 000 gal. per day, without appreciable loss of uniformity and still maintain intermittent operation of the siphon. The cycle of operation at average flow is slightly more than 20 min., of which time the nozzles are in operation about 8 min.

During the operation of the testing station, odors were observed at the aeration tank. These were characterized as similar to the animal den odors at the "Zoo". The intensity of such odors from the large aerating tank and with the weaker sewage from the starch works has not yet been determined. However, provision has been made for housing the aerating tanks and for burning the gases if necessary.

*Costs.*—The cost of the aeration plant is given in Table 18.

TABLE 18.—COST OF AERATION PLANT.

| Item. | Amount. |
|---|---|
| Contract 5 | $142,785.71 |
| Blowers | 16,000.00 |
| Switchboard | 3,397.00 |
| Oil filter equipment | 496.00 |
| Air meter | 943.00 |
| Air filter | 295.00 |
| Aeration plates | 2,800.00 |
| Dosing tank equipment | 2,550.00 |
| Additional piping | 1,351.35 |
| Blower house | 44,663.00 |
| Wrecking dosing tank | 1,100.00 |
| Total | $216,381.06 |

The unit prices of the major items were stated in the contract as follows:

| | | |
|---|---|---|
| Earth excavation | $0.50 | per cu. yd. |
| Rock excavation | 5.00 | " " " |
| Reinforcing steel | 0.053 | per lb. |
| Concrete in floors and footings | 12.25 | per cu. yd. |
| Concrete in walk and conduits | 20.00 | " " " |
| Miscellaneous steel and wrought iron | 0.13 | per lb. |
| Miscellaneous iron castings | 0.13 | " " |
| Cast-iron pipe, bell and spigot | 0.037 | " " |
| Cast-iron specials, bell and spigot | 0.093 | " " |
| Cast-iron specials, flanged | 0.132 | " " |
| Subway grating | 1.65 | per sq. ft. |

## Conclusion

The project at Decatur up to the final stages of operation has required nearly fifteen years of investigation and construction effort. Portions of the work, such as the organizing of the Sanitary District, and the treatment of the starch works sewage, were unusual and required careful and gradual working out. The persistence of the effort is noteworthy and is due in large measure to the continued service of the Board of Trustees, one of whom has remained throughout the entire life of the Sanitary District. It is the hope of the writers that the foregoing record will be useful in the consideration of other sewage disposal problems.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

# PAPERS AND DISCUSSIONS



# WATER SUPPLY PROBLEMS OF A DESERT REGION

By William E. Rudolph,* M. Am. Soc. C. E.

## Synopsis

This paper treats of some of the water supply problems of a section of the Atacama Desert of Northern Chile—the sources of water, the nature of the waters and the losses to which they are subject, and the works which have been installed for impounding and transporting water. Some notes are added on the utilization of surplus water resources for irrigation and for hydro-electric power, presenting conditions peculiar to arid regions.

## General

The water supply problems of desert regions are peculiar. Not only is precipitation scant in these arid lands, but the losses of water from evaporation and seepage, as well as contamination by absorption of salts, are factors which seriously affect the available supply.

Desert regions comprise nearly one-quarter of the earth's land surface. Some have developed considerable economic importance through the discovery of mineral deposits which the very aridity of the climate has preserved. Among the latter is the Atacama Desert of Northern Chile, where the world-famous nitrate fields are located, and also important bodies of copper ore. Here, a number of inland industrial centers have grown up, as well as several prosperous seaports, among the latter being Antofagasta and Iquique, cities of more than 50 000 population.

## Sources of Water of the Atacama Desert

Situated between two rain barriers, the Humboldt current and the Western Cordillera of the Andes, Northern Chile has little rainfall. At elevations below 10 000 ft., it averaged only 0.04 in. per year prior to 1925.† It is little

Note.—Written discussion on this paper will be closed in February, 1929.

* With Green Muth Bldg. Corporation, Rockville Center, N. Y.

† During the years 1925, 1926, and 1927, the rainfall has been greater, due perhaps to the supplanting of the cold northbound Humboldt current by the warmer southbound El Nino current during the summer of 1925. Such change in climate is probably of a transient character.

wonder that the Desert of Atacama is one of the most barren of the earth's surfaces.

Above an elevation of 10 000 ft. there are light showers during the summer (January to April) of most years. A single stream reaches the Pacific throughout the year—the Rio Loa (Fig. 1)—which has a maximum flow of 130 cu. ft. per sec., decreasing to 67 cu. ft. per sec., as it crosses the desert (Table 1). Yet this little stream supplies the Antofagasta nitrate pampa, the copper plant at Chuquicamata, and the City of Antofagasta, and contributes to the operation of three railroads and to the irrigation of farm lands.

In this latitude the Andes form two distinct ranges, the Cordillera Occidental and the Cordillera Oriental. Separating them is a region of high plateaus without surface drainage. The strip of land sloping to the Pacific, as well as portions of the high plateaus, form part of Chile.

The Cordillera Occidental (Fig. 1) consists of a uniformly high surface of folded and uplifted rocks, upon which volcanic masses have formed peaks and ranges. After the main range of the Cordillera Occidental had been raised to its present height, the forces of volcanism appear to have shifted toward the west, where subsidiary and lower chains of volcanic cones were built up. In but few places did they cut off drainage to the Pacific, but they did cause the formation of large lakes which gradually filled up with sediments, in some places to depths of perhaps 2 000 ft.

The waters which feed the Loa System originate primarily from summer snows and rains which fall in the high Cordillera region. These form a number of small streams (Fig. 2), most of which flow only during the heat of the day. The streams disappear within the sediment-filled lakes. Along the western bounds of these basins, at the lower points of the secondary ranges, the waters emerge as springs (Fig. 3), below which they have cut deep spectacular canyons through the volcanic formations to the lands beyond. By acting as natural reservoirs, the basins regulate the tributaries of the Loa and thus maintain a fairly constant flow throughout the day, season, and year.

East of the main range of the Cordillera Occidental are a number of wet basins without apparent outlet. Although subject to heavy evaporation, the lakes never dry up, for their water-sheds, being reached by moist east winds, experience no prolonged droughts. As they lie well above the Pacific draining region on the western side of the intervening range, there must be drainage from one to the other, probably along contacts of the older rocks and the newer volcanic topping which form this range. Thence, the subterranean waters flow directly into the basins of sediments, from which the streams of the western slopes draw most of their water. Thus, it would appear that the streams draining to the Pacific are fed from waters of the moister eastern slopes even when their local supply fails.

## Stream Flow Measurements

Recording stream flow in the Atacama Desert, particularly at the headwaters of the feeders, was a work fraught with difficulty. Changes in river beds caused by flood flows permitted leakage at the sides and below weirs. Several times complete destruction followed cloudbursts. At current-meter

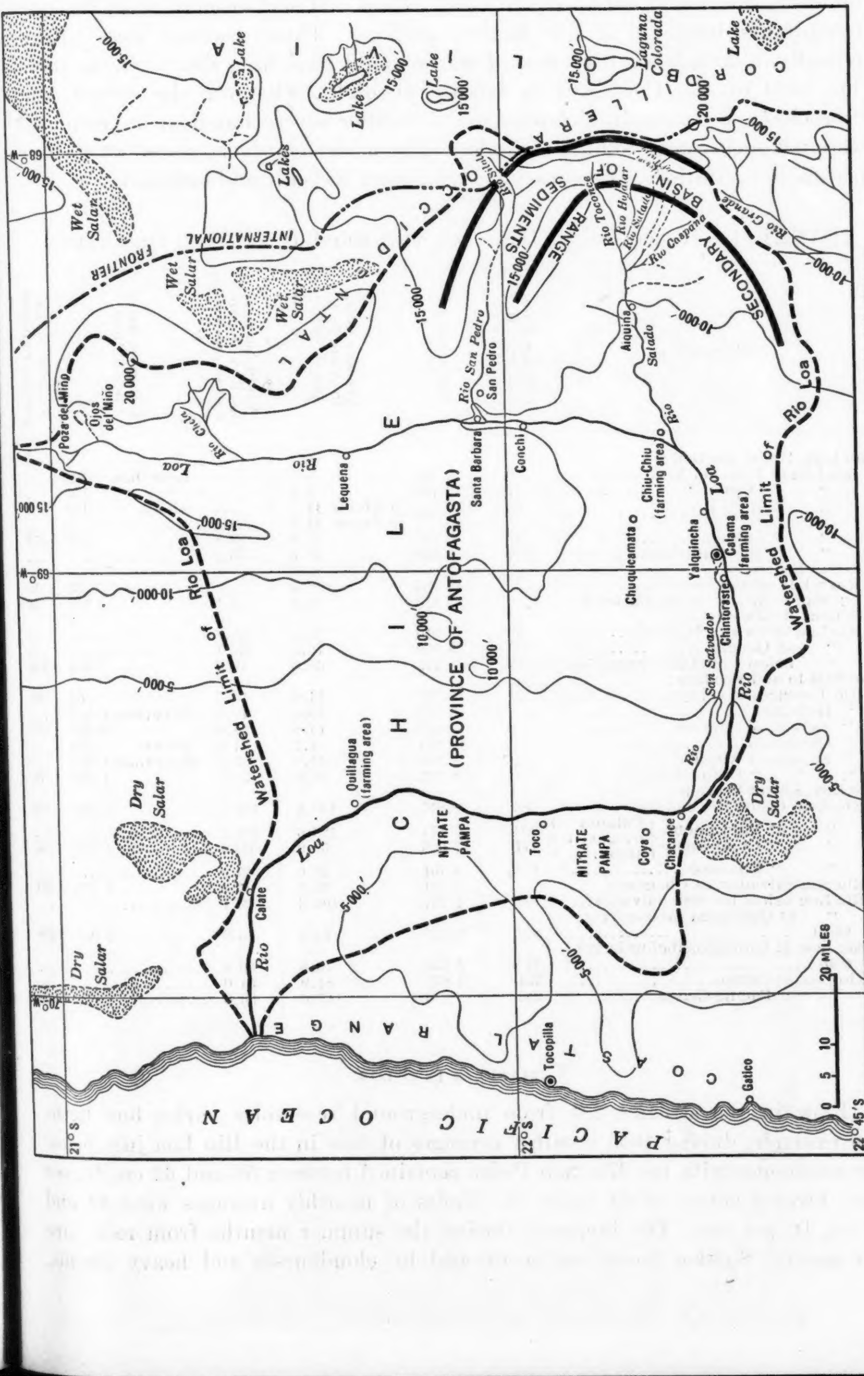


FIG. 1.—MAP OF CHILE BETWEEN LATITUDE 21° 0' AND 22° 45' SOUTH.

stations the configuration of the river bed was so often subjected to alteration by high water that constant checking was necessary during the rainy season.

Observation stations were of necessity located in isolated regions, a day's muleback journey apart in many cases. Food and shelter were poor, the only inhabitants being in a few Indian pueblos. These Indians were always friendly, although the diversion of water would have been disastrous to them. The cold of the Cordillera is intense at night, whatever the season, and "soroche", or mountain sickness, was a further severe handicap to men who were not accustomed to high altitudes. The native observers placed at stations had their limitations, and supervision was both difficult and expensive.

TABLE 1.—HYDROLOGICAL DATA FOR THE RIO LOA AND ITS TRIBUTARIES.

| Streams. | Distance from source of Rio Loa, in miles. | Elevation, in feet. | Average flow, in cubic feet per second. | Minimum normal flow, in cubic feet per second. | Chlorine, parts per million. | Sulfate radical, parts per million |
|---|---|---|---|---|---|---|
| **Rio Loa, Upper Section:** | | | | | | |
| Rio Loa at Pozo del Miño.......... | 0 | 12 800 | 0.8 | ..... | Less than 100 | ... |
| " " Ojos " ........... | 2 | 12 713 | 8.1 | ..... | " " 100 | ... |
| " " Rio Chela.............. | 19 | 11 860 | Above 11.7<br>Below 17.0 | .....<br>..... | About 100<br>............... | ...<br>... |
| " " Lequena.............. | 37 | 10 843 | 21.5 | 19.8 | 146 | 178 |
| " " Santa Barbara.......... | 57 | 9 810 | 45.9 | 35.3 | ............... | ... |
| **Rio San Pedro:** | | | | | | |
| Rio Siloli, at sources.............. | ... | 14 163 | 12.4 | ..... | 35 | 10 |
| " San Pedro, at Santa Barbara... | ... | 9 810 | 19.2 | 14.1 | 195 | 120 |
| **Rio Loa, Middle Section:** | | | | | | |
| Rio Loa below Rio San Pedro........ | 57 | 9 810 | 65.1 | 55.0 | ............... | ... |
| " at Conchi.............. | 62 | 9 560 | 70.6 | 61.1 | ............... | ... |
| " below Chiu Chiu Irrigation. | 86 | 8 215 | 60.0 | 49.4 | 403 | 144 |
| **Rio Salado and Feeders:** | | | | | | |
| Rio Toconce, at springs............ | ... | 13 287 | 14.0 | 13.5 | 62 | 50 |
| " Hojolar.................... | ... | 13 878 | 10.6 | 10.0 | More than 1 500 | ... |
| " Salado, at geysers.............. | ... | 14 167 | 14.8 | 14.0 | 4 201 | 48 |
| " Caspana.................... | ... | 12 224 | 4.2 | 4.0 | About 100 | ... |
| " Salado, at Aiquina........ ...... | ... | 9 580 | 45.9 | 37.1 | More than 1 500 | ... |
| " " " Chiu Chiu.......... | ... | 8 215 | 70.6 | 51.2 | 1 548 | 76 |
| **Rio Loa, Lower Section:** | | | | | | |
| Rio Loa, below Rio Salado......... | 86 | 8 215 | 130.6 | 109.5 | 1 126 | 115 |
| " at Yalquincha (Calama irrigated area) | 104 | 7 871 | 123.6 | 102.4 | .... ........... | ... |
| " " Chintoraste (Calama irrigated area)..... | 111 | 7 005 | 59.0 | 30.0 | 1 2[illegible]2 | 131 |
| " " Chacance.............. | 151 | 4 101 | 76.6 | 45.9 | ............... | ... |
| Rio San Salvador, at Chacance..... | ... | 4 101 | 26.0 | 22.2 | 1 996 | 264 |
| Rio Loa below Rio San Salvador... | 151 | 4 101 | 102.6 | 68.1 | ............... | ... |
| " at Quillagua (above irrigation)................... | 206 | 2 733 | 84.8 | 45.9 | 2 088 | 288 |
| Rio Loa, at Quillagua (below irrigation)................... | 210 | 2 500 | 70.6 | 30.0 | ............... | ... |
| Rio Loa, at Calate.............. | 231 | 1 877 | 81.9 | 51.0 | ............... | ... |
| " " Pacific Ocean........... | 252 | 0 | 67.0 | 36.0 | ............... | ... |

## VARIATIONS IN FLOW

Flow in the streams fed from underground reservoirs varies but little. For example, during 1913 monthly averages of flow in the Rio Loa just below the confluence with the Rio San Pedro remained between 56 and 62 cu. ft. per sec. Over a period of $3\frac{1}{2}$ years, the limits of monthly averages were 54 and 71 cu. ft. per sec. The increases during the summer months from rains are not great. Sudden floods are occasioned by cloudbursts and heavy storms,

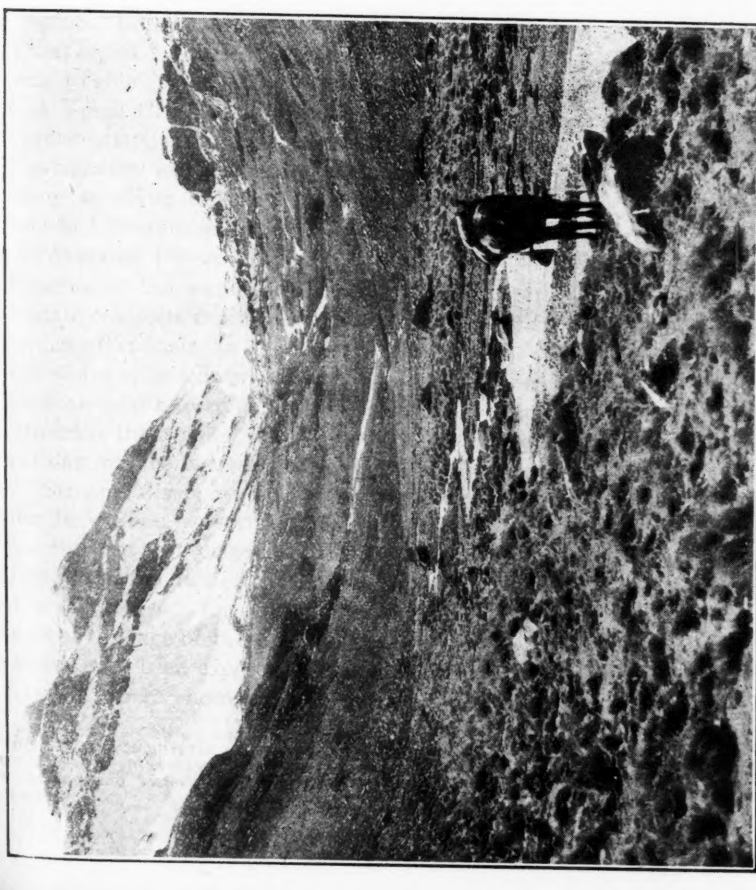

FIG. 2.—VIEW OF SMALL STREAMS ORIGINATING FROM SUMMER SNOWS AND RAINS IN THE ANDES IN CHILE.

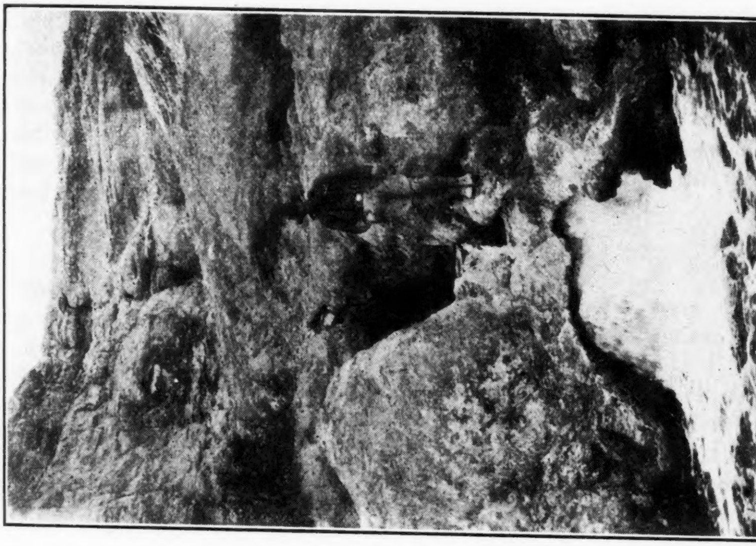

FIG. 3.—UNDERGROUND FLOW EMERGING AS SPRING IN THE ANDES IN CHILE.

but high water seldom lasts for more than 3 or 4 hours. In February, 1925, the volume of the Loa below the confluence with the Salado, near Chiu-Chiu, rose to more than 15 000 cu. ft. per sec. for a few hours, this being 115 times the normal flow and probably the highest in the recent history of the river.

During the irrigating season, from September to March, losses of water in the Loa below Chiu-Chiu are heavy, as may be seen from the minimum normal flows in Table 1.

## Losses of Water

The flow and usability of waters of the region suffer from (1) losses from evaporation; (2) losses from seepage; and (3) absorption of salts. These three important factors will be discussed separately.

*Evaporation.*—Waters which enter interior drainage basins, mere evaporating pans, are lost in evaporation. Flows of the Loa and other river systems are also subject to high evaporation whenever they spread out to form marshes. Where water-works have been installed, the construction of narrow masonry canals to drain these swamp lands, or to carry the waters past them, has been necessary.

The magnitude of these losses is due in part to the extreme dryness of this region. Comparison of the evaporation observed in the Atacama Desert with that given by Ivan E. Houk, M. Am. Soc. C. E., for U. S. Reclamation projects (Table 2) show that rates in the Chilean desert are generally higher than at Yuma Citrus, the dryest of Mr. Houk's stations. While the writer's data were compiled partly by the Chilean Government and partly by industrial companies, and, therefore, probably are not of the same high degree of accuracy as those of Mr. Houk, they are nevertheless significant.* The stations at Chacance and Chuquicamata are representative of the arid sections of the Atacama Desert.

Eventually, the water requirements of the Northern Chilean desert may necessitate complete drainage of the lakes on the Bolivian side of the frontier, by means of tunnels, to avoid evaporation losses. The costs of such works are not so apt to be a drawback as the international complications.

*Seepage.*—So numerous are the lava flows which have obstructed drainages and diverted them from one course to another that a detailed study of such formations may be necessary in order to determine the origin of a particular flow. Streams disappear below the surface and, later, re-appear, sometimes greater in volume, sometimes less. That underground flows are often large in quantity has been demonstrated by the increases observed in stream flow at sections where no run-off is received. Isolated water holes, ofttimes in the most arid parts of the desert, furnish further proof of such underground flows. On the face of the Coastal Range escarpment beside the Pacific, where the strata have been exposed by fracture, springs are frequent—although this is a region where showers occur perhaps once in a year. Part of the flow

* None of the Chilean observations was based on the use of floating pans. Inasmuch as the high evaporation rates of the Atacama Desert are due to high average wind velocity rather than to high average temperature, it would seem that there would be far less difference between land and water-pan measurements in this case than at the stations mentioned by Mr. Houk.

of these springs is doubtless due to condensations from the afternoon clouds which envelop the coastal cliffs, but more, perhaps, comes from underground passages extending a hundred miles or more inland to the Cordillera region.

TABLE 2.—COMPARISON OF EVAPORATION RECORDS OF UNITED STATES RECLAMATION PROJECTS* AND OF ATACAMA DESERT, CHILE.

| Station. | Years. | Annual evaporation, in inches. | Mean temperature, in degrees Fahrenheit. | Mean wind velocity, in miles per hour. | Remarks. |
|---|---|---|---|---|---|
| Yuma Citrus, Ariz. ..... | 1921-23 | 122.81 | 70.7 | 3.03 | Station 8 miles southwest of Yuma on desert mesa. |
| Yuma Citrus, Ariz. ..... | 1921 | 135.70 | 70.7 | 3.03 | Maximum during 3 years, 1921 to 1923. |
| Elephant Butte, N. Mex | 1917 | 109.70 | 61.1 | 4.52 | Maximum during 7 years, 1917 to 1923. |
| San Pedro ............... | 1917 and 1918 | 117.15 | About 50 | About 10 | In mountain region subject to annual precipitation of about 1.7 in. |
| Chuquicamata ........ | Various observations, 1917-27 | 144.00 | " 58 | 9.81 | In arid desert, very little precipitation. |
| Calama.......... | 1917 | 113.40 | " 57 | Ranging between 6 and 8 | Within irrigated area, very little precipitation. |
| Chacance .............. | 1917 and 1918 | 181.00 | " 65 | | In arid desert, rainfall minimum, about once in 2 years. |
| Quillagua ............ | 1917 | 119.00 | " 65 | | Within irrigated area, rainfall minimum, about once in 2 years. |

* From paper by Ivan E. Houk, M. Am. Soc. C. E., entitled, "Evaporation on United States Reclamation Projects," *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 266.

The underlying rock formations are of such structure as to cause loss of much of the water passing through them. Some of the waters that wander away through limestone passages or surface sediments, are certain to return to the main channel because of the water-tight flooring of hard crystalline rocks. On the other hand where such waters have access to underlying stratified rocks of a permeable nature, heavy losses result, due to the steep slopes which facilitate subterranean flow direct to the ocean. It is difficult to trace waters lost in the pervious strata of sandstone because the overlying mantles of detritus are sometimes hundreds of feet in thickness.

Thus far, losses due to seepage have not called for considerable attention because of the moderate water requirements. With increased exploitation of the mineral resources of this region the engineer must face the task of reclaiming these waters or of preventing their escape. As far as the writer knows, no deep borings have been undertaken to reach the underground flows, which probably become concentrated within buried drainage systems of the past.

*Absorption of Salts.*—Some of the volcanic barriers through which the flows pass contain considerable salts of uncertain origin. Perhaps they were caused by decomposition of volcanic tuff due to fumarolic waters, such as still exist in the region. Again, they may be from vast beds of sodium chloride which crystallized out in basins as the continent was elevated more than two miles out of the sea, later to be covered with volcanic materials. Whatever

the cause, many of the waters that are chemically quite pure near their sources, acquire so much salt while passing through volcanic formations, particularly at the lower boundaries of the basins of sediments, that they cease to be potable.

Chemical concentrations are highest in waters of geysers and boiling springs. The geysers of the Tatio initiate a flow of 14.8 cu. ft. per sec., with a chlorine content of 4 200 parts per million. The Chilean Government once considered diverting this river to an evaporating basin outside the Rio Loa System, but a study of the economics of such an undertaking caused it to be abandoned.

Seldom are waters of the desert so chemically pure as those ordinarily used in regions of normal rainfall. Only at the upper reaches of the Loa System (Table 1) is the water low in chlorides and sulfates. In the United States, waters containing chlorine in excess of 20 parts per million are not desirable because of corrosion to boilers and plumbing fixtures. In Northern Chile chlorine tolerance for potable waters and those used for generating steam is usually set at 50 parts per million, and the Chilean Government has stretched the point slightly by including within this category waters of the Rio Loa System which have as much as 62 parts per million.

Whereas waters used in steam boilers may be treated for sulfates and carbonates, the writer knows of no practicable way of eliminating chlorine. The waters of the Rio Toconce, containing 62 parts per million of chlorine, have been used for a number of years in locomotives. The Antofagasta-Bolivia Railroad Company at one time used the water of the Rio San Pedro having 195 parts per million of chlorine, but had to obtain other supplies. For drinking, waters similar to those of the Rio San Pedro are apt to have a mild purgative effect. Those having as much as 1 000 parts per million of chlorine are not fit to drink. The people of the Village of Quillagua, although very poor, purchase drinking water from the Railroad Company at the equivalent of 1¼ cents (U. S. currency) per gal., rather than use the brackish water of the Loa.

For irrigation the Chilean Government classifies as good, waters not exceeding 200 parts per million in chlorine. In general, only the Indian communities near the head-waters of the region have such water. Nevertheless, good crops of alfalfa, corn, and some vegetables are raised with water of from two to ten times this maximum.

Streams which spread out to form marshes are naturally high in salinity. The salt content of most of the waters of the region is also increased 50% or more during the summer season, when the rains in the upper sections dissolve saline efflorescences.

In the basins without outlet, the lakes have a widely diversified chemical content, including alkalis and borax. The impotability of many of these waters is attested by the bones of animals along the shores. In other lakes, as the Laguna Colorada, the brackish water forms a large pool in the center, separated by crystallizations of salts from the outside waters, which are potable.

## Intakes for Potable Water Supply

In view of what has been said about evaporation and seepage losses as well as the acquisition of soluble salts, apparently potable water intakes might best be located at the upper sources of the feeders, that is, at the snow drainages, before they disappear into the reservoirs of sediment. This is seldom feasible, however, because of (*a*) the high altitude and general inaccessibility of such places; (*b*) the intermittent character of flow due to freezing; and (*c*) the fact that in very dry years there may be no flow at all.

In a few cases snow drainage water, although flowing underground, has been piped by building subterranean dams—such works being limited to narrow valleys. At others, the waters have been taken from rivers at elevations between 11 000 and 14 000 ft. Tunnels driven into sandstone formations to collect and divert ground-waters to intakes have also been resorted to, but they seldom yield as much as natural springs. For example, the fifteen canals of the Pica Region, Province of Tarapaca, discharge but little more than 1 cu. ft. per sec., despite their 8 miles of length.

An advantageous location for the intake is at the springs where the waters issue from the natural resevoirs. They may have already dissolved some salts, but not in large quantity. Small canals carry the water to the settling basin at the intake of the pipe line (Fig. 4).

Where waters are removed from streams, the diversion dam and intake chamber require merely sufficient height to keep the pipe line from drawing air. Constant cleaning is necessary, for not only is there considerable sediment, but also floating matter, chiefly pebbles of pumice, washed from the ground by the rains. Pollution by sewage seldom calls for consideration in locating a potable water intake, for the water sources are rarely inhabited.

## Pipe Lines

Pipe lines depend on gravity in the upper sections and pumping stations in the lower. The pipes are nearly always of steel or cast iron; wood-stave lines, although cheaper, are hardly safe in regions of the desert where water is scarce and scruples are few. One structure was made of hard durable metal on purpose "to resist destruction by travelers who ordinarily take such objects as targets for revolver or rifle shots."

These lines are laid on the ground with a covering of about 15 in. of soil, except at the joints. This protects them against temperature changes which sometimes amount to 80° Fahr. within 12 hours in Northern Chile. The pipe line full of water is not as a rule affected by temperature, but when a break occurs difficulties may ensue. Pipe lines are covered as they are laid, using expansion joints to take up such temperature stresses until the earth covering becomes compact.

In many cases the water is conveyed for long distances, the supply of the City of Antofagasta traveling 230 miles. Some of the lines are operated under but little pressure, open tanks being located at intervals. Others, which cross low valleys, necessitate pressures as high as 1 000 lb. per sq. in.

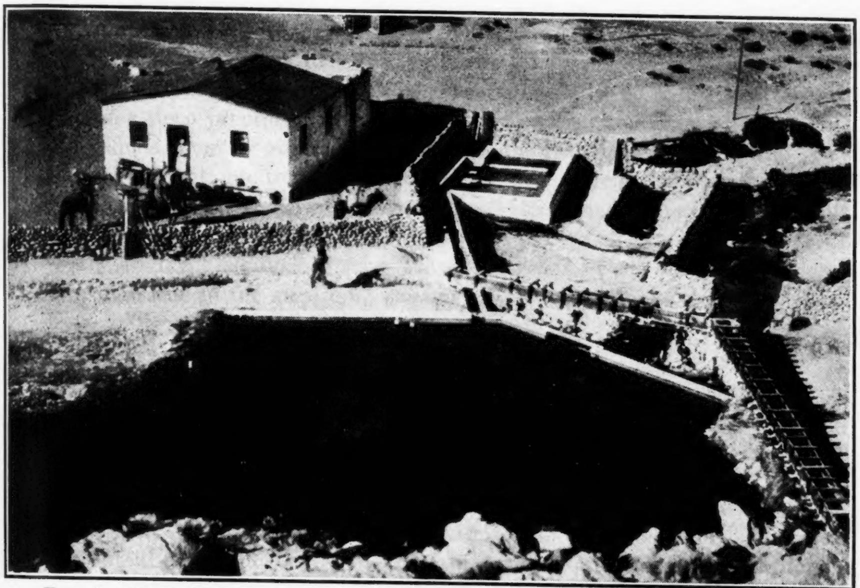

FIG. 4.—VIEW SHOWING SMALL CANAL CARRYING WATER TO SETTLING BASIN AT INTAKE OF PIPE LINE.

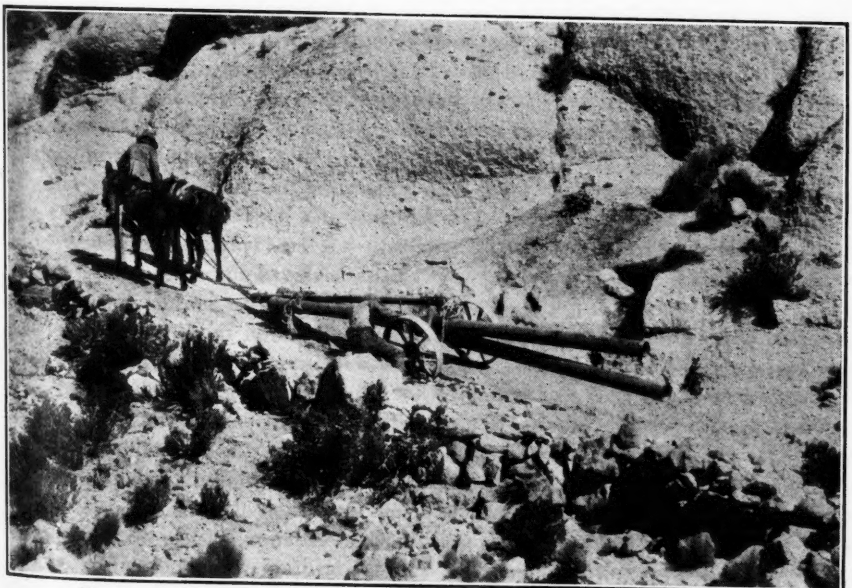

FIG. 5.—METHOD OF TRANSPORTING MATERIALS FOR PIPE LINES IN CHILE.

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Design of pipe lines ofttimes presents complications. Transportation of materials must be considered in the location (Fig. 5), for, in country covered with lava flows and cut by deep canyons, delivery costs may far exceed the value of the pipe.* Also, important is the hydraulic grade, particularly in the section just below the intake where the line must generally be located within the bounds of a deep precipitous canyon. Here, construction expenses are high. Yet to reduce them by resorting to a minimum grade for the pipe line is apt to be short-sighted, due to the tendency of the hydraulic grade to fall below the level of the pipe after several years of operation, with diminished capacity and other troubles resulting. The writer recalls one instance wherein it was necessary to lay "looping" lines after eight years of operation, in order to bring a pipe line to the minimum capacity for which it was designed.

## Operation

The high chemical content of the water is apt to make pipe corrosion problems more acute in desert regions than under similar conditions in humid lands, for sulfates and chlorides, usually so abundant, aid the attack on iron. In Northern Chile, analysis of nodular incrustations found in a pipe line showed a composition of 0.8% sulfate radical and 0.2% chlorine, a concentration which is quite remarkable considering that only 50 and 62 parts per million of these radicals occurred in the water itself. Ground-waters of regions of recent volcanic activity, where rainfall is scant, are apt to contain large quantities of carbon dioxide and hydrogen sulfide, both gases being agents of corrosion. The warmth of spring waters (often as high as 75° Fahr. throughout the year) also contributes to their corrosive properties.

Because of the corrosive quality of these waters, it is highly important that air be excluded from pipe lines. When the hydraulic grade line falls below the level of the pipe, even for transient periods as when scouring is done, water wasted, or branch lines fed, air is admitted and conditions favorable to corrosion are set up.

For removing sand and sediments, sand traps are best, as they do not cause the impairments so often attendant upon scouring. Valves for emptying a pipe line are necessary, as for discharging the water when repairs are in progress; but when such valves are to be used for scouring, their locations as well as the manner of their operation call for careful study. One company used floating balls of hollow metal for ridding its line of sand. The ball was made about 1½ in. smaller in diameter than the pipe. When its progress was checked by accumulations of sand the pressure rose and the obstruction was removed. At the same time the ball was not so large as to endanger the line by completely blocking the flow. Its passage could be checked by the noise which it made—in fact, a dog was trained to follow and bark at it.

Automatic air release valves, located at summits where the pressure is low, serve to eject some of the air entering the line. With cold non-corrosive water, and steady flow maintained under pressures greater than atmospheric,

* Mules had been used to a great extent in the past for haulage and distribution. In more recent years, motor trucks, tractors, and trailers have effected considerable savings over the older methods, despite the initial expenses for road building. In soft sandy portions of the desert, caterpillar trailers have been used to good advantage.

deterioration of a long-distance water main will be slow. With corrosive water, it may be well to install works for de-ærating the water or rendering it non-corrosive at the source.

Some protective coatings appear sufficiently substantial to safeguard a line against corrosion. However, until a coating is found combining low cost with durability it will probably be more economical to take measures to exclude free oxygen from pipe lines.

The type of joint is another matter which is important. Too frequently the common screw coupling provides most favorable conditions for storing air, with consequent chemical injury.

## Other Means of Transportation of Water

Many of the villages are supplied from tank cars, the inhabitants making daily trips to the station to fill their cans. Another means of distribution is the mule cart, with its daily rounds of nitrate district villages as well as smaller pueblos. In more isolated districts, one sees mules and burros carrying potable water, often for 20 or 30 miles. Whereas 26 gal. per day per inhabitant within cities, and half that quantity outside of cities, are the quotas on which the Chilean Government bases concessions for drinking water supplies in its desert provinces, the actual use of water is generally less. This is due to the extreme poverty of the people. Large industrial companies have inaugurated the practice of furnishing free water to workmen and their families, as well as providing sanitary toilets and bath houses for their use.

## Irrigation

A potent argument for conservation of water resources in desert regions under development lies in the needs of agriculture. Where food is scarce and expensive, with increments of population continually being brought in to work at mines and industries, all surplus water is required for irrigation. This applies not only to flows lost through seepage and useless evaporation, but also to those that reach the sea.

In Northern Chile large water resources are still being lost or wasted. Indian dwellers of the highlands have installed irrigation canals, marvels of ingenuity, within the canyons of the upper drainages, but these produce only sufficient for their owners' needs. Farming centers in sections of lower altitude cultivate alfafa in quantities, but only small quantity of foodstuffs. Meanwhile, imported food fills the shelves of every store, and that from Central and Southern Chile has to be transported about 1 000 miles.

That irrigation can produce successful crops has been demonstrated at Chiu-Chiu, Calama, and Quillagua, within the Rio Loa Basin. Yet this river pours 36 cu. ft. per sec. into the sea during the farming season. Once there was a project—work had actually been begun, in fact—to enlarge the cultivated area at Quillagua. The plan came to naught when the earthquake of 1879 destroyed some of the completed canals. It can be only a question of time before foreign capital interested in the nitrates and minerals of this district will take the matter in hand.

## Hydro-Electric Power

In a region where water is scarce it would be thought that hydro-electric power possibilities would be of little consequence. This does not necessarily follow, for a meager supply may be balanced to some extent by the high head available.

With present low prices for fuel oil, developments of the hydro-electric resources of the Loa and adjacent basins do not appear attractive. Yet such potential installations, costing between $450 and $500 per h.p., require no considerable rise in fuel costs before they assume importance. A few water-power plants, generating about 500 h.p. per installation, have been in operation in the lower sections of the Rio Loa for some years, in connection with the nitrate industry.

As a source of energy, however, the harnessing of the Loa and adjacent waters has hardly been begun. Projects at the upper feeders of these rivers are unique in that they call for series of power houses using the same water again and again. The Rio Salado and its tributaries may be utilized through a head of 5 000 ft. in 35 miles above its confluence with the Loa, developing about 8 000 h.p. Discharges from springs of as little as 0.1 cu. ft. per sec., when at the higher elevations, represent important potential energy and in many cases are well worth diverting to the canals.

Storage reservoirs are seldom advantageous, due to the constancy of stream flows and to the high rate of evaporation. The diversion dam at the intake of the canal requires only sufficient height either to meet the requirements of diurnal variations, due to freezing of tributaries in the upper regions, or to regulate flow below sections when irrigation is carried on during the daylight hours, in the lower regions.

A contributing factor in making these projects so expensive is the porous nature of many of the rock formations. Lined canals are necessary for the most part, particularly in view of the scarcity of water. The volcanic rock formations of the mountain region are peculiar in that the hardest and most durable often occur at the surface, so that erosion has formed overhanging cliffs at canyons, with massive fallen fragments among the talus beneath. Needless to say such conditions call for expensive construction to safeguard water-works.

Only above an elevation of 12 000 ft. need power-house roofs be designed for snow load. Frost does not affect foundations carried 2 ft. below ground.

Some water supply pipe lines have been designed to furnish energy for operating power plants in this region. A small emergency plant, installed near the outlet, is quite certain to justify its cost, provided the pipe line is not overstressed under the increased pressures.

## Distillation of Ocean Waters

Distillation of ocean waters is another source of supply. The seaport of Tocopilla obtains its water in this way, at a cost of $0.60 (U. S. currency) per ton and more. With a prosperous period for the Chilean nitrate industry and consequent growth of this port, a pipe line from the Cordillera region would seem necessary.

This method is out of the question for the nitrate and mining centers as these are situated at elevations of 3 000 ft., or more. A drinking supply for a large number of these establishments is obtained by distillation of brackish river waters.

## CONCLUSION

It has been seen how the projecting of water-works in a desert region may involve a great variety of problems, from the utilization of snow waters at 3 miles above sea level to the distillation of waters of the ocean itself. So scant is the supply in the Atacama Desert of Northern Chile that, with expansion of industry, the engineer is apt to be delving within the realms of geology and chemistry in order to find ways of conserving the quantity and quality of the water available. Perhaps the discussion will bring to light further developments under similar conditions.

**AMERICAN SOCIETY OF CIVIL ENGINEERS**
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# PAPERS AND DISCUSSIONS

**This Society is not responsible for any statement made or opinion expressed in its publications.**

# BAFFLE-PIER EXPERIMENTS ON MODELS OF PIT RIVER DAMS

## Discussion*

By J. C. Stevens, M. Am. Soc. C. E.

J. C. Stevens,† M. Am. Soc. C. E. (by letter).‡—In November, 1927, the writer conducted some experiments on a one-twelfth size model of the diversion dam for the Leaburg hydro-electric development. This is the second of a series of hydro-electric power plants to be constructed on McKenzie River by the City of Eugene, Ore.

The object of the experiments was to ascertain (1) the effect the proposed dam would have on flood heights above it; (2) the type of baffles on the apron most effective in preventing scour; (3) the most satisfactory relation between height of dams and height of gates; and (4) the manner in which the flood-gates should be operated, with the apron and baffles selected to avoid scouring the river bed. This development consists essentially of a diversion dam to raise the river level 20 ft., a canal of 2 200 sec-ft. capacity, 5 miles in length, and a power house where this flow is utilized under 90 ft. of head, thereby creating 20 000 h. p.

The Leaburg Diversion Dam will consist of a low overflow weir with roller type of gates on the crest, 100 ft. long and 12½ ft. high, supported by piers. There are three such gates and, in addition, a 30 by 18-ft. sluice-gate, for auxiliary regulation.

Provision is made for passing 90 000 sec-ft. over the dam with all gates open. The dam must not raise flood water more than 2½ to 3 ft. higher than would occur in the river without it. A field laboratory was constructed to which water could be drawn under control from the Walterville Canal, which supplies City Plant No. 1.

Fig. 58 shows a plan of the laboratory and the dam and baffles used for the six series of experiments made. Fig. 59 shows the setting of the

* Discussion of the paper by I. C. Steele and R. A. Monroe, Members, Am. Soc. C. E. continued from August, 1928, *Proceedings*.

† Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

‡ Received by the Secretary, June 12, 1928.

model and baffles in the flume. It was specified that stock for Pieces *A*, *B*, and *C* be cut to the stock sizes shown and then soaked in water completely submerged for at least ten days prior to milling. After milling, the pieces were submerged in water until used. The object was to secure pieces accurately dimensioned when thoroughly saturated. It was permitted to use No. 1 clear white pine, cedar, or redwood, but all must be of the same stock.

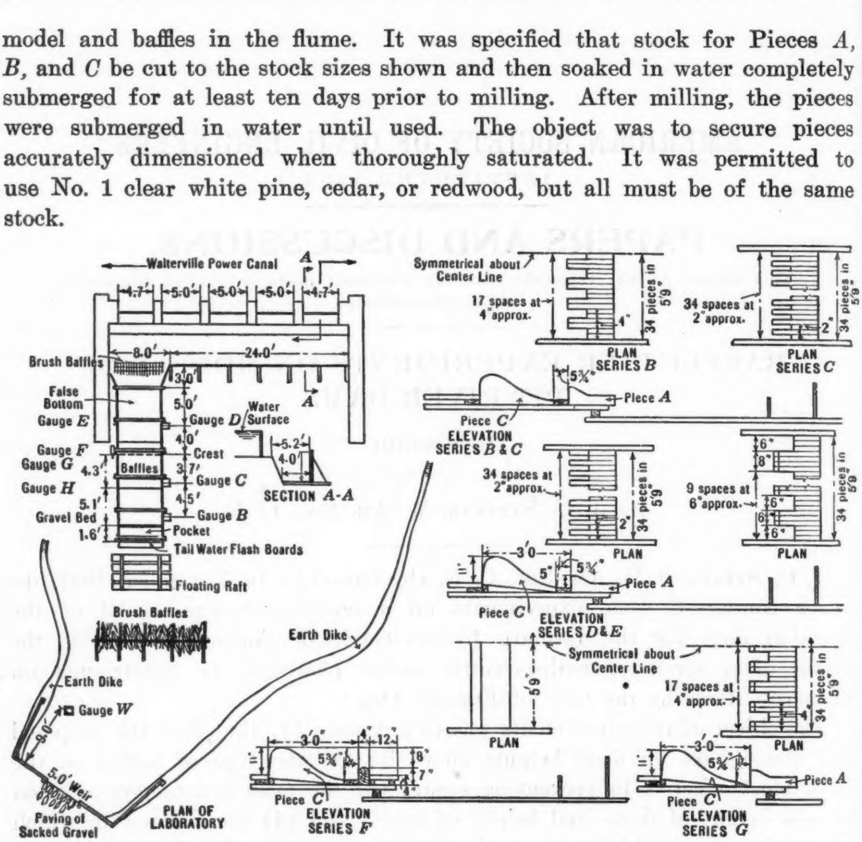


FIG. 58.—PLAN OF HYDRAULIC LABORATORY AND MODEL, LEABURG POWER DEVELOPMENT.

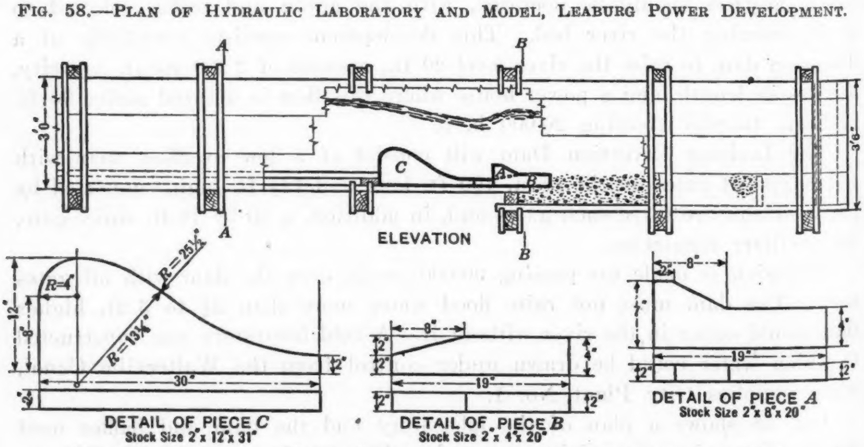


FIG. 59.—DETAILS OF MODEL DAM AND BAFFLES, WITH ASSEMBLY IN FLUME, LEABURG POWER DEVELOPMENT.

Water was drawn from the wasteways of the power canal into a 6 by 4-ft. timber flume. It was quieted above the dam by brush baffles; then it was

passed over the model dam and gravel bed, dropped into a small pond, and thence, through another set of brush baffles and over the 5-ft. rectangular measuring weir.

After passing the apron with its baffles, the water flowed over a bed composed of sand and small gravel graded similarly, but sized as nearly as possible to one-twelfth those of the bed of the prototype. How nearly the prototype gravel bed was imitated in the model is shown in Fig. 60. The river gravel tested was a composite of five 100-lb. samples taken from the river bed at the dam site. The model gravels were made up from screened and recombined river sands and gravels.

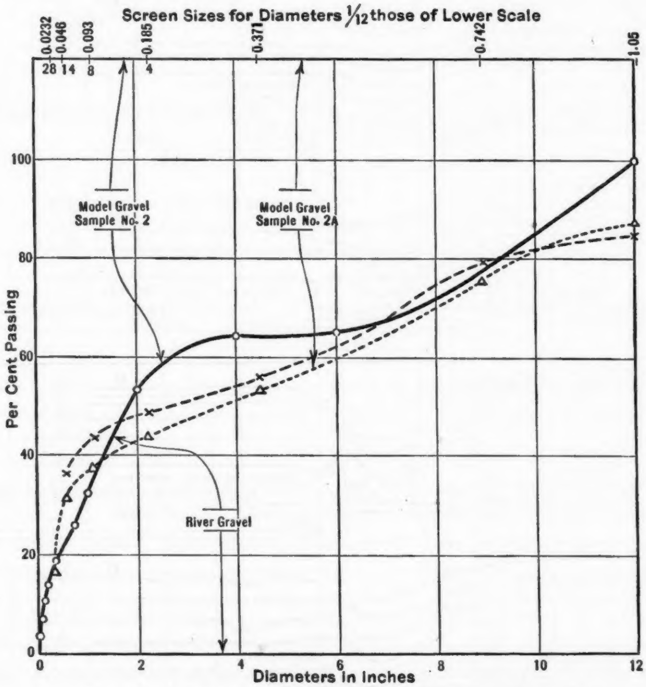


FIG. 60.—GRADING OF RIVER AND MODEL GRAVELS.

As shown on Fig. 59 the model dam and baffles were made up of laminations or blocks of 2-in. lumber thoroughly soaked before milling. By the use of such laminations various combinations of widths in the baffles and in the spaces between them were secured as shown in Fig. 61. Thirty-four laminations fitted the 69-in. width of flume exactly.

During tests the height of tail-water was regulated by stop-logs in the end of the flume, to levels corresponding to the elevations of the water surface in the river below the dam for the corresponding flows.

The data obtained at each test consisted of complete water surface profiles above and below the dam, and the amount of scour, if any, on the gravel bed. The latter was obtained by levels before and after tests, and

by the observance of gravel washed into a pocket left between the gravel bed and the regulating stop-logs. Fig. 61 shows the water surface profiles. Test numbers are shown at the left end of the profile lines, while the figures

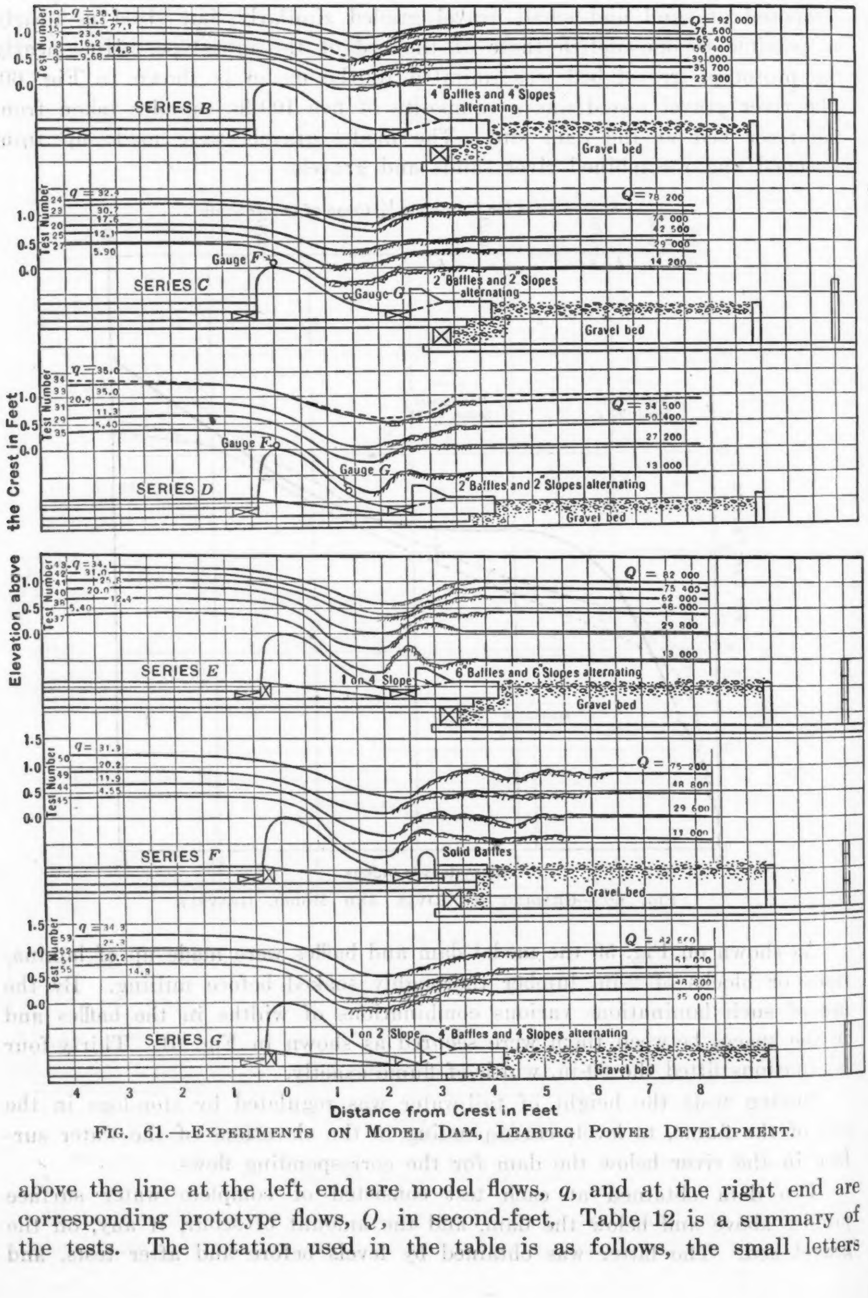


FIG. 61.—EXPERIMENTS ON MODEL DAM, LEABURG POWER DEVELOPMENT.

above the line at the left end are model flows, $q$, and at the right end are corresponding prototype flows, $Q$, in second-feet. Table 12 is a summary of the tests. The notation used in the table is as follows, the small letters

signifying values for the model and the capital letters those for the prototype:

$h$ = head on crest of dam, from Gauge $D$, in feet.
$q$ = flow, in second-feet, on model.
$e_1$ = up-stream energy head, referred to crest, in feet.
$e_2$ = down-stream energy head, referred to crest, in feet.
$i$ = lost energy head, in feet.
$q'$ = flow per foot of crest of model.
$c$ = coefficient in formula, $q' = c\, e_1^{\frac{3}{2}}$.
$e_1'$ = energy head, in feet, for given flow if dam is not submerged, calculated from average values of $c = 3.75$ for Series $B$ and $C$ and for $c = 3.80$ for Series $D$, $E$, $F$, and $G$.
$h'$ = head on crest, in feet, for given flow if dam is not submerged.
$d$ = measured depth on crest, in feet.
$d_c$ = critical depth on crest, in feet, calculated from $d_c = 0.098\, q^{\frac{2}{3}}$.
$f$ = pressure head on crest, in feet, measured by Gauge $F$.
$Q$ = river flow, in second-feet, at Leaburg Dam.
$I$ = energy head lost, in feet.
$H$ = head water elevation = $h$ converted, in feet.
$H'$ = head-water elevation, in feet, if dam is not submerged.
$D$ = elevation of water over crest = $d$ converted, in feet.
$D_c$ = elevation for critical flow over crest = $d_c$ converted, in feet.
$F$ = elevation of pressure head on crest = $f$ converted, in feet.

Fig. 62 shows various hydraulic curves pertaining to the experiments, all referred to the prototype.

The various gauges used are shown in Fig. 58, and are described in Table 13. In addition to the gauges in Table 13 a profile board consisting of 2 by 6-in. timber was set on edge lengthwise over the center of the flume. Distances were marked thereon up stream and down stream from the crest of the dam. Elevations were taken on each foot mark, and the longitudinal profile of the water surface was determined by measuring, with a level rod, the distances to the water surface from the top edge of the profile board.

Of the experiments made, five series were with open baffles; that is, square-faced piers with spaces between them, and one series with a solid baffle or weir the full width of the apron.

An examination of the profiles (Fig. 61) will show that in all cases where the open baffles were used the hydraulic jump occurs well on the apron. For low flows it occurs just over the baffles. As the flow was increased, it tended to move down stream, but remained on the apron.

The solid baffle in Series $F$ invariably produced a double jump; the second occurred over the gravel bed, and was attended with more or less scour. The advantage of open baffles in controlling the position of the jump and in preventing scour is apparent, as it permits the use of a much shorter apron.

TABLE 12.—SUMMARY OF EXPERIMENTS ON MODEL OF LEABURG DIVERSION DAM AND CONVERSION TO PROTOTYPE QUANTITIES.

| Series and test. | On Model. | | | | | | | | | | | | | | | On Prototype, | | | | | | | | | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Head, $h$, in feet. | Flow, $q$, in second-feet. | Tail-Water. | | Submergence, percentage. | Energy loss. | | | For unsubmerged flow. | | | | On crest. | | | Head, $H$, in feet. | Flow, $Q$, in second-feet. | Tail-Water. | | $I$, in feet. | $H'$, in feet. | $D$, in feet. | $D_c$, in feet. | $F$, in feet. | |
| | | | Actual, in feet. | Normal, in feet. | | $e_1$, in feet. | $e_2$, in feet. | $i$, in feet. | $q'$, in second-feet. | $c$. | $e_1'$, in feet. | $h'$, in feet. | $d$, in feet. | $d_c$, in feet. | $f$, in feet. | | | Actual, in feet. | Normal, in feet. | | | | | | |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) | (11) | (12) | (13) | (14) | (15) | (16) | (17) | (18) | (19) | (20) | (21) | (22) | (23) | (24) | (25) | (26) |
| *B*- 7 | 0.97 | 23.4 | 0.79 | 0.73 | 82 | 1.06 | 0.90 | 0.16 | 4.06 | ...... | 1.05 | 0.97 | 0.85 | 0.80 | .... | 739.1 | 56 400 | 737.0 | 737.0 | 1.92 | 739.1 | 737.7 | 737.1 | ...... | No scour of gravel bed. |
| 8 | 0.69 | 14.8 | 0.37 | 0.43 | 54 | 0.74 | 0.45 | 0.29 | 2.56 | ...... | 0.77 | 0.72 | 0.58 | 0.59 | .... | 735.8 | 35 700 | 731.9 | 732.7 | 3.48 | 736.1 | 734.4 | 734.6 | ...... | |
| 9 | 0.56 | 9.68 | 0.15 | 0.18 | 27 | 0.59 | 0.21 | 0.38 | 1.68 | 3.74 | 0.59 | 0.56 | 0.44 | 0.44 | .... | 734.2 | 23 300 | 729.3 | 729.6 | 4.56 | 734.1 | 732.8 | 732.8 | ...... | |
| 13 | 0.76 | 16.2 | 0.51 | 0.49 | 67 | 0.82 | 0.59 | 0.23 | 2.81 | 3.80 | 0.83 | 0.77 | 0.63 | 0.63 | .... | 736.6 | 39 000 | 733.6 | 733.4 | 2.76 | 736.7 | 735.1 | 735.1 | ...... | |
| 15 | 1.10 | 26.9 | 0.93 | 0.94 | 84 | 1.21 | 1.06 | 0.15 | 4.68 | ...... | 1.16 | 1.05 | 1.00 | 0.88 | .... | 740.7 | 64 900 | 738.7 | 738.7 | 1.80 | 740.1 | 739.5 | 738.1 | ...... | Some fines washed into pocket.* |
| 16 | 1.02 | 26.4 | 0.80 | 0.92 | 78 | 1.13 | 0.95 | 0.18 | 4.60 | ...... | 1.15 | 1.04 | .... | 0.87 | .... | 739.7 | 63 600 | 737.1 | 738.5 | 2.16 | 739.9 | ...... | 737.9 | ...... | |
| 17 | 1.00 | 25.6 | 0.68 | 0.89 | 68 | 1.11 | 0.83 | 0.28 | 4.45 | 3.80 | 1.12 | 1.01 | .... | 0.85 | .... | 739.5 | 61 600 | 735.7 | 738.1 | 3.36 | 739.6 | ...... | 737.7 | ...... | |
| 18 | 1.23 | 31.5 | 1.07 | 1.10 | 87 | 1.36 | 1.24 | 0.12 | 5.48 | ...... | 1.27 | 1.13 | 1.14 | 0.98 | .... | 742.2 | 76 500 | 740.3 | 740.7 | 1.44 | 741.0 | 741.2 | 739.3 | ...... | Slight scour. |
| 19 | 1.37 | 38.1 | 1.23 | 1.32 | 90 | 1.53 | 1.41 | 0.12 | 6.80 | ...... | 1.49 | 1.31 | 1.29 | 1.11 | .... | 743.9 | 92 000 | 742.3 | 743.4 | 1.44 | 743.2 | 743.0 | 740.8 | ...... | Considerable scour. |
| *C*-20 | 0.79 | 17.6 | 0.57 | 0.56 | 72 | 0.85 | 0.66 | 0.19 | 3.06 | ...... | 0.87 | 0.80 | 0.67 | 0.66 | .... | 737.0 | 42 500 | 734.3 | 734.1 | 2.28 | 737.1 | 735.5 | 735.4 | ...... | No scour. |
| 22 | 1.17 | 31.2 | 0.96 | 1.09 | 82 | 1.30 | 1.13 | 0.14 | 5.43 | ...... | 1.28 | 1.14 | .... | 0.97 | .... | 741.5 | 75 000 | 739.0 | 740.4 | 1.68 | 741.1 | ...... | 739.1 | ...... | Some fines washed into pocket. |
| 23 | 1.21 | 30.6 | 1.06 | 1.07 | 88 | 1.33 | 1.20 | 0.13 | 5.33 | ...... | 1.26 | 1.12 | 1.12 | 0.96 | .... | 742.0 | 73 700 | 740.2 | 740.2 | 1.56 | 740.9 | 740.9 | 739.0 | ...... | |
| 24 | 1.29 | 32.8 | 1.17 | 1.13 | 90 | 1.41 | 1.31 | 0.10 | 5.62 | ...... | 1.31 | 1.17 | 1.21 | 0.99 | .... | 743.0 | 77 500 | 741.5 | 741.0 | 1.20 | 741.5 | 742.0 | 739.4 | ...... | |
| 25 | 0.65 | 12.1 | 0.28 | 0.30 | 43 | 0.69 | 0.35 | 0.34 | 2.10 | 3.70 | 0.68 | 0.64 | 0.50 | 0.52 | 0.89 | 735.3 | 29 000 | 730.9 | 731.1 | 4.09 | 735.2 | 733.5 | 733.7 | 732.2 | No scour. |
| 27 | 0.42 | 5.90 | −0.06 | −0.06 | 0 | 0.48 | −0.02 | 0.45 | 1.02 | (3.75) | 0.42 | 0.41 | 0.31 | 0.32 | 0.28 | 732.5 | 14 200 | 726.8 | 726.8 | 5.40 | 732.1 | 731.2 | 731.3 | 730.9 | |
| *D*-29 | 0.63 | 11.2 | 0.03 | 0.01 | 5 | 0.65 | 0.09 | 0.56 | 1.95 | 3.75 | 0.64 | 0.62 | 0.51 | 0.49 | 0.30 | 738.0 | 27 000 | 730.9 | 730.6 | 6.72 | 737.9 | 736.6 | 736.4 | 734.1 | No scour. |
| 31 | 0.87 | 20.8 | 0.46 | 0.44 | 52 | 0.93 | 0.56 | 0.37 | 3.62 | ...... | 0.97 | 0.91 | 0.74 | 0.74 | 0.32 | 740.9 | 50 000 | 736.0 | 735.8 | 4.45 | 741.4 | 739.4 | 739.4 | 734.3 | Few fines washed into pocket. |
| 32 | 0.86 | 20.5 | 0.54 | 0.43 | 63 | 0.92 | 0.63 | 0.29 | 3.56 | ...... | 0.96 | 0.90 | 0.74 | 0.73 | .... | 740.8 | 49 400 | 737.0 | 735.7 | 3.38 | 741.3 | 739.4 | 739.3 | ...... | |
| 33 | 1.27 | 34.8 | 0.93 | 0.95 | 75 | 1.39 | 1.08 | 0.31 | 6.06 | ...... | 1.37 | 1.25 | 1.06 | 1.05 | 0.55 | 745.7 | 84 000 | 741.6 | 742.1 | 3.72 | 745.5 | 743.2 | 743.1 | 737.0 | Some scour. |
| 34 | 1.33 | 34.8 | 0.99 | 0.95 | 82 | 1.44 | 1.22 | 0.22 | 6.06 | ...... | 1.37 | 1.25 | 1.10 | 1.05 | .... | 746.5 | 84 000 | 742.3 | 742.1 | 2.64 | 745.5 | 743.7 | 743.1 | ...... | |
| 35 | 0.37 | 4.50 | −0.49 | −0.43 | 0 | 0.38 | −0.45 | 0.83 | 0.78 | ...... | 0.35 | 0.34 | 0.27 | 0.27 | 0.25 | 734.9 | 10 800 | 724.6 | 725.4 | 10.0 | 734.6 | 733.7 | 733.7 | 733.5 | No scour. |
| *E*-37 | 0.36 | 4.50 | −0.53 | −0.43 | 0 | 0.37 | −0.48 | 0.85 | 0.78 | ...... | 0.35 | 0.34 | 0.25 | 0.27 | .... | 734.8 | 10 800 | 724.1 | 725.4 | 10.2 | 734.6 | 733.5 | 733.7 | ...... | No scour. |
| 38 | 0.65 | 12.3 | 0.06 | 0.06 | 9 | 0.68 | 0.13 | 0.55 | 2.13 | 3.70 | 0.68 | 0.65 | 0.52 | 0.52 | 0.33 | 738.3 | 29 600 | 731.2 | 731.2 | 6.61 | 738.3 | 736.7 | 736.7 | 734.5 | |
| 40 | 0.87 | 20.0 | 0.40 | 0.41 | 46 | 0.93 | 0.50 | 0.43 | 3.48 | 3.86 | 0.94 | 0.89 | 0.71 | 0.72 | .... | 740.9 | 48 200 | 735.3 | 735.3 | 5.16 | 741.2 | 739.0 | 739.1 | ...... | |
| 41 | 1.05 | 25.8 | 0.70 | 0.64 | 67 | 1.13 | 0.81 | 0.32 | 4.50 | ...... | 1.12 | 1.04 | 0.87 | 0.86 | 0.47 | 743.1 | 62 000 | 738.9 | 738.1 | 3.84 | 743.0 | 740.9 | 740.8 | 736.1 | Some scour. |
| 42 | 1.18 | 30.9 | 0.86 | 0.83 | 73 | 1.28 | 0.99 | 0.29 | 5.38 | ...... | 1.26 | 1.16 | 1.02 | 0.96 | 0.52 | 744.7 | 74 200 | 740.8 | 740.5 | 3.48 | 744.4 | 742.7 | 742.0 | 736.7 | |
| 43 | 1.28 | 34.0 | 0.98 | 0.94 | 77 | 1.39 | 1.11 | 0.28 | 5.91 | ...... | 1.34 | 1.25 | 1.11 | 1.03 | 0.60 | 745.9 | 81 800 | 742.3 | 741.7 | 3.36 | 745.5 | 743.8 | 742.9 | 737.7 | Considerable scour. |
| *F*-44 | 0.65 | 11.9 | 0.01 | 0.04 | 0 | 0.68 | 0.08 | 0.60 | 2.07 | 3.72 | 0.66 | 0.63 | 0.52 | 0.51 | 0.32 | 738.3 | 29 600 | 730.7 | 731.1 | 7.20 | 738.1 | 736.7 | 736.6 | 734.3 | Some scour.† |
| 45 | 0.37 | 4.70 | −0.44 | −0.42 | 0 | 0.37 | −0.41 | 0.78 | 0.82 | ...... | 0.36 | 0.36 | 0.31 | 0.28 | 0.25 | 734.9 | 11 300 | 725.2 | 725.6 | 9.38 | 734.8 | 734.1 | 733.8 | 733.5 | No scour. |
| 48 | 0.65 | 12.3 | 0.05 | 0.05 | 8 | 0.68 | 0.12 | 0.56 | 2.14 | 3.82 | 0.68 | 0.65 | .... | 0.52 | 0.33 | 738.3 | 29 600 | 731.1 | 731.2 | 6.72 | 738.3 | ...... | 736.7 | 734.5 | Considerable scour.† |
| 49 | 0.89 | 20.2 | 0.48 | 0.42 | 48 | 0.95 | 0.52 | 0.43 | 3.50 | 3.81 | 0.95 | 0.89 | 0.70 | 0.73 | 0.35 | 741.2 | 48 800 | 735.7 | 735.5 | 5.16 | 741.2 | 738.9 | 739.3 | 734.7 | |
| 50 | 1.20 | 31.2 | 0.84 | 0.84 | 70 | 1.30 | 0.96 | 0.34 | 5.43 | ...... | 1.27 | 1.17 | 1.03 | 0.97 | 0.52 | 744.7 | 75 000 | 740.6 | 740.5 | 4.09 | 744.5 | 742.9 | 742.1 | 736.7 | |
| *G*-51 | 0.88 | 20.2 | 0.46 | 0.42 | 52 | 0.94 | 0.55 | 0.39 | 3.52 | 3.86 | 0.95 | 0.89 | 0.75 | 0.73 | 0.33 | 741.1 | 48 800 | 736.0 | 735.5 | 4.69 | 741.2 | 739.5 | 739.3 | 734.5 | No scour. |
| 52 | 1.01 | 25.3 | 0.62 | 0.62 | 61 | 1.09 | 0.74 | 0.35 | 4.40 | 3.85 | 1.10 | 1.02 | 0.86 | 0.85 | 0.41 | 742.6 | 61 000 | 738.0 | 737.9 | 4.21 | 742.7 | 740.8 | 740.7 | 735.4 | No scour. |
| 53 | 1.28 | 34.3 | 0.96 | 0.95 | 75 | 1.39 | 1.10 | 0.29 | 5.95 | ...... | 1.35 | 1.28 | 1.12 | 1.04 | 0.59 | 745.9 | 82 600 | 742.0 | 741.8 | 3.48 | 745.3 | 743.7 | 743.0 | 737.6 | Considerable scour. |
| 55 | 0.73 | 14.5 | 0.20 | 0.17 | 27 | 0.76 | 0.27 | 0.49 | 2.52 | 3.80 (3.80) | 0.76 | 0.73 | 0.58 | 0.58 | 0.32 | 739.3 | 35 000 | 732.9 | 732.5 | 5.89 | 739.2 | 737.5 | 737.5 | 734.3 | No scour. |

* Tests 15, 16 and 17 show effect of lowering tail-water. While adjusting flash-boards for Test No. 17, tail-water was lowered too far. This obliterated the jump and produced a smooth standing wave with trough over the gravel bed, which scoured the bed. No further scour resulted after tail-water was properly adjusted and jump re-established. † Scour due to double jump.

The arrangement of laminations used to produce various widths of baffles and openings had no effect that could be detected. This is to be expected since the retarding force is practically the same as long as the aggregate

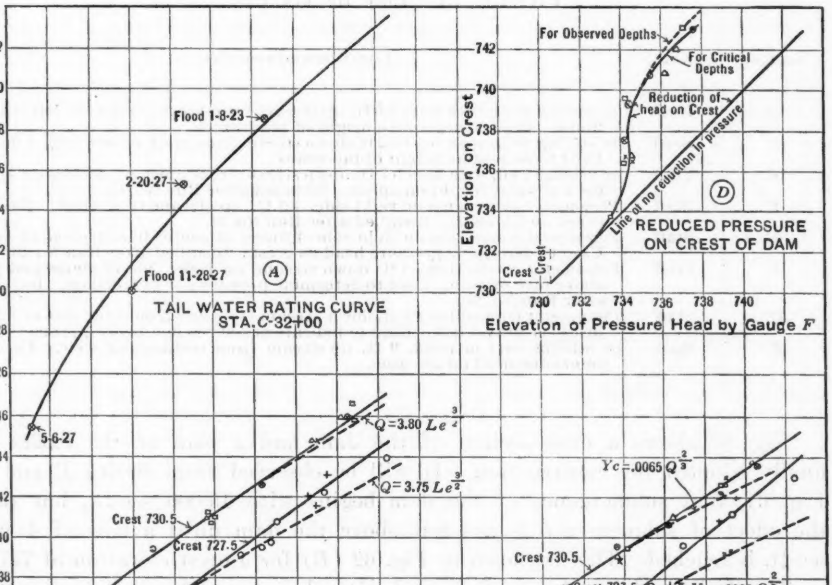


FIG. 62.—HYDRAULIC CURVES, LEABURG DIVERSION.

width of baffle and opening remains the same. For convenience of construction, baffles 4 ft. in width, with openings of like width between them, were adopted.

The upward slope of the openings between the baffles was 1 on 4 in all except Series *G*, in which a slope of 1 on 2 was used. No difference could be detected; hence, a 1 on 4 slope was adopted for ease of construction.

TABLE 13.—List of Gauges.

| Gauge. | Kind. | Location and remarks. |
|---|---|---|
| *D* | Hook | In stilling well, with inlet 4.0 ft. up stream from crest of dam on left side of flume. Used to determine height of head-water. |
| *B* | Staff | In stilling well, with inlet 8.2 ft. down stream from crest, on left side of flume. Used to determine height of tail-water. |
| *C* | Staff | In stilling well, with inlet 3.7 ft. down stream from crest on left side. Used for tail-water height on apron. Installed after Run No. 17. |
| *E* | Staff | Piezometer connection on right side, 4.0 ft. up stream from crest. Used as check on Gauge *D*. Installed after Run No. 24. |
| *F* | Staff | Piezometer connection on right side of flume at center line of crest of dam. Used to determine pressure head on crest. Installed after Run No. 24. |
| *G* | Staff | Piezometer connection 1.4 ft. down stream, on right side of flume and just above face of dam. Used to determine pressures on face of dam. Installed after Run No. 24. |
| *H* | Staff | Piezometer connection 4.5 ft. down stream from crest, on right side of flume just over gravel bed. Used to indicate pressures just below apron. |
| *W* | Hook | In stilling well in pond, 9 ft. up stream from rectangular weir. Used to determine head on the weir. |

Fig. 63 shows a cross-section of the dam and a plan of the baffles as finally adopted for construction. It will be observed from Series *B* and *C* Fig. 61, that submergence of the dam begins with 15 000 sec-ft., but that the effect of submergence is not felt above the dam until a flow of 40 000 sec-ft. is reached. This is shown in Fig. 62 (*B*) for a crest elevation of 727.5.

The fall in the water surface and also the energy head lost as water passes the dam are shown in Fig. 62 (*E*). For the maximum flow of 90 000 sec-ft. the head lost is only 1.5 ft.; for the more frequent flood of 40 000 sec-ft., the drop in water surface is 3.1 ft.; and the energy head lost is 2.6 ft.

Some interesting data were secured on the cavitation effect as water passed the crest of the dam. Gauge *F* was a piezometer gauge in the side of the flume in line with the center line of the crest of the dam as shown in Series *C* and *D*, Fig. 61. It therefore registered the pressure head of the water at this point. The profile readings gave the actual depth at the crest. The difference is the reduced pressure or cavitation effect produced by the change of direction of water as it passed the crest. These data are shown at (*D*) Fig. 61. The difference amounted to 6 ft. for the higher flows.

The increased pressure on the down-stream face of the dam was similarly obtained by the profiles and piezometer, Gauge *G*. This increase in pressure is shown at (*F*) Fig. 62.

During tests with flows corresponding to 40 000 sec-ft., or more, a lead pencil, if placed beneath the nappe on the face of the dam, would play up and down on the face. In some instances its range of travel would be almost over the entire face from near the crest almost to the baffles. There was undoubtedly an oscillation of the vacuum near the crest and the increase of pressure on the face, that accounted for this phenomenon. It could not be due to eddy motion between the baffles and the face, for in that case the

pencil would have risen into the nappe. Instead it remained on the face sliding and rolling along it.

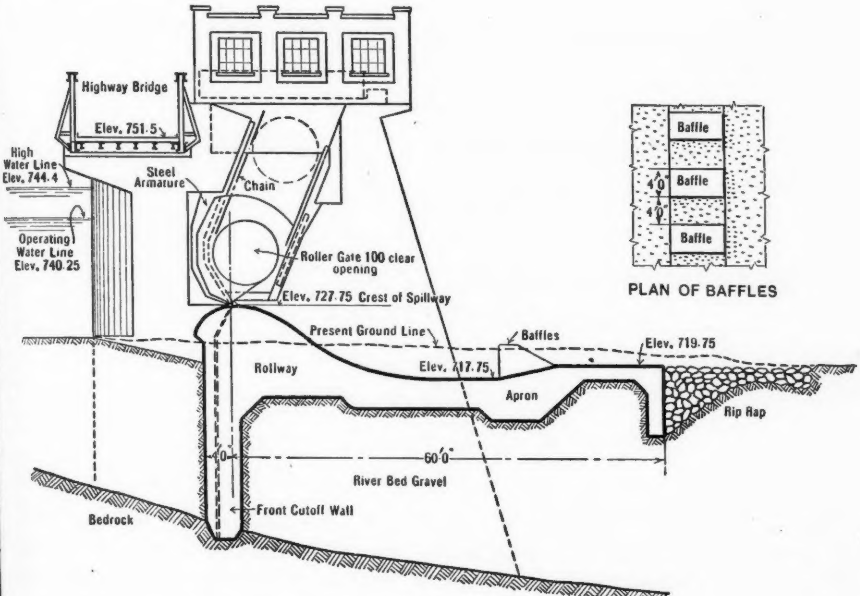


FIG. 63.—TYPICAL SECTION THROUGH SPILLWAY, LEABURG DAM.

During extreme floods the river gravels will be rolled along the bed whether or not there is a dam. If scour can be prevented at the immediate toe of the dam, it is the best that can be hoped. The experiments showed that with either type of baffles used, scour accompanied the formation of a second wave beyond the apron, and that no scour occurred immediately below the apron. With a large flow and the tail-water held considerably lower than will actually occur, the water passed over the baffles as clear and smooth as if they were not there. A standing wave formed a considerable distance below the apron, that scooped a broad depression in the gravel bed.

In practice, such a condition can only occur if one large gate were wide open during a flood. Considerable experimenting was done with various gate openings and as a result the plan will be adopted of opening the three roller-gates together, to maintain the desired head above the dam during floods. If this plan had not been adopted, it would have been necessary to increase the height of the baffles more nearly in conformity with those used on the Pit River dams.

The writer believes the type of open baffle used on the Leaburg Dam will be more effective in maintaining a hydraulic jump on the apron than the truncated or sharp-nosed deflectors and piers used for the Pit River dams, and that either type is far superior to a solid weir.

The authors did not try the type described herein, but the writer trusts that any future experiments will include the Leaburg type.

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# PAPERS AND DISCUSSIONS



## THE SCIENCE OF FOUNDATIONS—ITS PRESENT AND FUTURE

### Discussion*

BY ARTHUR M. SHAW, M. AM. SOC. C. E.

ARTHUR M. SHAW,† M. AM. SOC. C. E. (by letter).‡—This paper is quite unique in that it discusses clay and other soils in the same terms as have been used in the discussion of other materials of construction. While many experimental data have been compiled concerning the bearing power of soils, it ordinarily has not occurred to the engineer dealing with these widely used materials to investigate such properties as tensile strength, elasticity, and especially strength in shear, although the author shows that all these properties are of vital importance in at least some types of soils. The writer was particularly interested in the discussion of the relation of area to the supporting power of soils,§ having noticed the apparent inconsistent behavior of weak soils under loads of uniform weight per square foot, but varying materially in area covered.

The following discussion will be limited to muck soils, overlying soft clay, the type of soils commonly found in the "prairie" areas of the lower delta of the Mississippi River.

A number of years ago, it became necessary to place a fuel-oil tank convenient to a pumping plant on a reclamation project near New Orleans, La. The soil, of the type described, had been drained only partly and under a test load (placed on a 3 by 3-ft. platform) appeared to be safe for a load of approximately 500 lb. per sq. ft. A spread foundation was designed to limit the load to 400 lb. per sq. ft., but before the tank was quite three-quarters full, it began to settle unevenly and to an undesirable degree. Hasty shoring brought the tank to a vertical position, an occasional re-adjustment of the

* Discussion of the paper by Charles Terzaghi, M. Am. Soc. C. E., continued from August, 1928, *Proceedings*.

† Cons. Engr., New Orleans, La.

‡ Received by the Secretary, June 13, 1928.

§ *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2265.

shores being necessary to keep it plumb. After about six months of subdrainage, and compacting by the superimposed load, the soil was capable of supporting the full load of 400 lb. without material additional settlement. The total settlement was 2.5 ft.

On a similar reclamation project, experiments were made with the freshly drained muck soil to determine loads which might be used in the construction of light buildings for housing employees. A limit of 250 lb. was adopted and results indicated that this was about correct. General settlement was expected as a result of the drainage and shrinkage of the muck, but this was fairly uniform and did not cause any serious complications except in connection with one or two buildings which were provided with brick flues. These were carried on short posts (old pile-heads) which rested on a thin local stratum of sand only about 5 ft. below the surface. The considerable separation of grade of the living-room floor and the fireplace has resulted in a novel architectural effect. In a recent conversation the Superintendent (who has been in charge since the work started in 1917) advised that he now is using a load of 500 lb. per sq. ft. for small footings, but for large footings, he has found it necessary to adopt a considerably lower load limit.

In the construction of levees on similar foundations, the expedient of driving sheet-piles to prevent lateral movement or "bulging" of the muck has been adopted in local practice although it has been found that this is necessary only in extreme cases. Just as good results usually can be secured, and at much less cost, by constructing the levee in multiple layers and by adopting a design that will tend to increase the resistance of the soil to horizontal stresses. Fig. 37 illustrates the section and the method which the writer has used successfully in the construction of levees on exceptionally soft muck lands. The levee section was controlled by the limit of reach of available dredging equipment, as it is seldom practicable to secure a dredge in this section with a reach from the side of the hull in excess of 60 ft. By the use of longer boom dredges, such as were advocated by the writer a number of years ago,* a material increase in height of levees in such soils should be possible. A special type of dredge, which has been in use in California for many years, is well adapted to levee building in soft soils or under other conditions requiring an exceptionally long reach. Following are the principal dimensions of this dredge: Width of hull, 70 ft.; length of hull, 140 ft.; maximum reach from side of hull, 180 ft.; and nominal capacity of bucket, 6 cu. yd.

Repeated experiments have shown that both the spread base and the construction of levees in multiple operations have a beneficial effect in increasing the stability of muck and soft clay soils. With regard to the spread base, the gradually increasing weight, from toe of levee toward the crown, apparently compresses the muck in the zones subjected to only partial loading and renders it more resistant to lateral movement. The gradual application of the load in multiple layers permits the excess water to be expelled from the subsoils and increases their supporting power.

* "The Selection and Operation of Dredges", *Engineering Record*, December 16, 23, and 30, 1916.

In the lands referred to, there is a great variation in the weight of materials entering into levees, due to different proportions of muck top-soil and clay subsoil. There also is a wide range in weight of the muck soils, some containing a considerably larger proportion of silt than others. As would be expected, the muck soils dry out rapidly after being placed in embankments, above water level, but the "sharkey" clay gives off its water content very slowly. Excavations into clay levees more than two years old have shown a "gummy" condition at points 5 ft. above soil-water level.

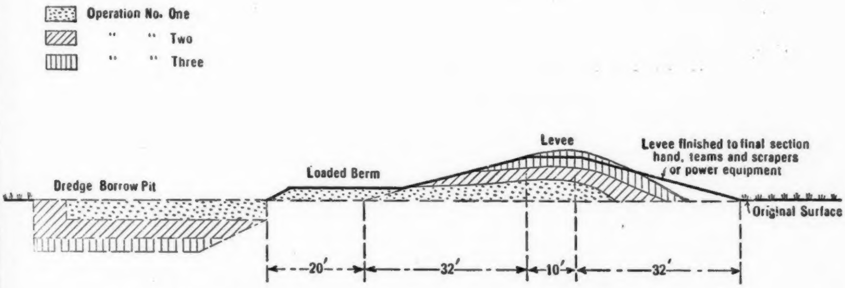


FIG. 37.—CONSTRUCTION OF A LEVEE BY MULTIPLE OPERATIONS.

While concentrated loads, such as those of embankments, may result in failure of the lower strata by a horizontal movement, fills, over large areas, may be constructed to a considerable height on muck soils, with no danger of settlement beyond that caused by the squeezing out of some of the water. The writer has had occasion to make many hydraulic fills over soft muck lands, the added material frequently being clay or sand, but he has never experienced difficulty from excessive subsidence in such cases, provided the fills extended to hard ground or were tapered off gradually at the edges, thus securing the gradual loading mentioned in the foregoing as being desirable for levee construction in similar soils.

Most engineers who have had experience in the construction of railroad embankments across river bottoms have known of localities where embankments 6 to 10 ft. in height could be built without complications, but where a fill of 15 or 20 ft. in height would subside (sometimes suddenly). This subsidence usually would be accompanied by an upheaval of the natural surface, 20 to 50 ft. beyond the toe of the slope. Two such phenomena occurred during the construction of the Illinois Central Railroad from Fort Dodge, Iowa, to Omaha, Nebr., in 1900, one at a point about ten miles north of Council Bluffs, Iowa, and the other in the South Omaha cut-off line, near the bluffs in East Omaha. In the latter case service on the near-by tracks of the Missouri Pacific Railroad was interrupted by the upheaval.

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# PAPERS AND DISCUSSIONS



## ANALYSIS OF ARCH DAMS BY THE TRIAL LOAD METHOD

### Discussion*

BY MESSRS. J. L. SAVAGE AND IVAN E. HOUK, AND FRED A. NOETZLI.

J. L. SAVAGE† and IVAN E. HOUK,‡ MEMBERS, AM. SOC. C. E. (by letter).§—Several important changes and improvements have been made by the U. S. Bureau of Reclamation in its methods of analyzing arch dams since this paper was prepared. These modifications are too elaborate to be adequately explained in a brief discussion. However, it seems advisable at this time to state briefly the nature of the changes that have been made and to discuss their effect on the analysis of the Gibson Dam now being built on the Sun River Project of Montana, this being one of the dams mentioned by the authors.

In the original analysis of the Gibson Dam the sides of the cantilevers were assumed to be parallel; the temperature curve shown in Fig. 4,‖ which was based on an incomplete study of limited experimental data, was used as a basis for calculating temperature stresses in the arches; the effect of shear in the arches was neglected; the foundation and abutment rock was assumed to be rigid; and no arch action was assumed to take place until the increasing water load had reduced the up-stream deflections of the cantilevers to zero. In the revised calculations, the sides of the cantilevers were taken as radial; a new temperature curve, based on a careful study of many actual measurements of concrete temperature variations, was used as a basis for calculating temperature stresses in the arches; the effect of shear in the arches was considered; the foundation and abutment deformations were taken into account in both cantilever and arch elements; and transmission of load to the abutments by arch action was assumed to begin as soon as the reservoir begins to fill.

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* Discussion of the paper by C. H. Howell, M. Am. Soc. C. E., and the late A. C. Jaquith, Esq., continued from August, 1928, *Proceedings*.

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‡ Research Engr., U. S. Bureau of Reclamation, Denver, Colo.

§ Received by the Secretary, June 30, 1928.

‖ *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 66.

Thus far, the writers have not taken into account the saturation of concrete near the up-stream face of the dam, flow of concrete, effect of Poisson's ratio, transmission of load by tangential shear between arches, effect of vertical shear between cantilevers, effect of variations in the modulus of elasticity, the distribution of stress due to twisting of the horizontal and vertical elements, some of the effects introduced through foundation and abutment deformations, or the errors introduced through the assumption of a linear stress distribution. However, all these conditions are being investigated, and the writers hope, eventually, to be able to estimate the magnitude of some of their effects as regards the stresses within the structure and its safety against failure through any cause.

The fact that the sides of the cantilever should be taken as radial vertical planes instead of parallel vertical planes was recognized at the time the original analysis of the Gibson Dam was made. While it was realized that proper allowance for this condition would tend to increase the bending of the cantilevers under given loads, thus tending to increase the stresses at the down-stream face of the dam, it was not believed that the effect of the correction would be as important as it was later found to be, especially as regards bending where the cantilevers are already cracked. Of course, the increased bending and accompanying increased stress under a given load is not the final effect on the analysis of the structure. The arches must bend over to meet the increased deflections of the cantilevers, and in order to do so they will carry some of the load formerly allotted to the cantilevers. Consequently, the net result of considering the cantilever sides as radial instead of parallel is a new load distribution with slightly increased deflections and stresses.

Fig. 48 shows the increase in down-stream deflection of the crown cantilever at the Gibson Dam, also the increase in tension area, caused by considering the sides of the cantilever as radial instead of parallel, the load being the same in each calculation. It will be noticed that, while the proportionate increase in total tension area is not as great as might be expected, the decrease in width of uncracked base and the increase in down-stream deflection of the cantilever, resulting from the corrected method of calculation, are both important. The latter is seen to be practically double the original deflection at all elevations.

Fig. 49 compares the original temperature curve of Fig. 4 with the revised curve used in the latest analysis of the Gibson Dam. In determining the revised curve the variations in water temperature at different depths in the reservoir were taken into account, as well as the maximum and minimum air temperatures at the down-stream face of the dam. It will be noticed that the new curve is higher than the old at all points. Consequently, its use tends to give higher temperature stresses. Since the curve gives the total annual range in concrete temperatures, only one-half the values shown need be used in calculating arch stresses, it being assumed that the contraction joints will be grouted, or closing plugs poured, when the concrete temperatures are equal to, or less than, the mean annual temperature of the dam. Since the expansion accompanying a rise in temperature works against the water load, only the contraction accompanying the decrease in temperature need be considered.

The new curve was not extended farther at the left of the diagram since 15 ft. is the minimum thickness of the Gibson Dam.

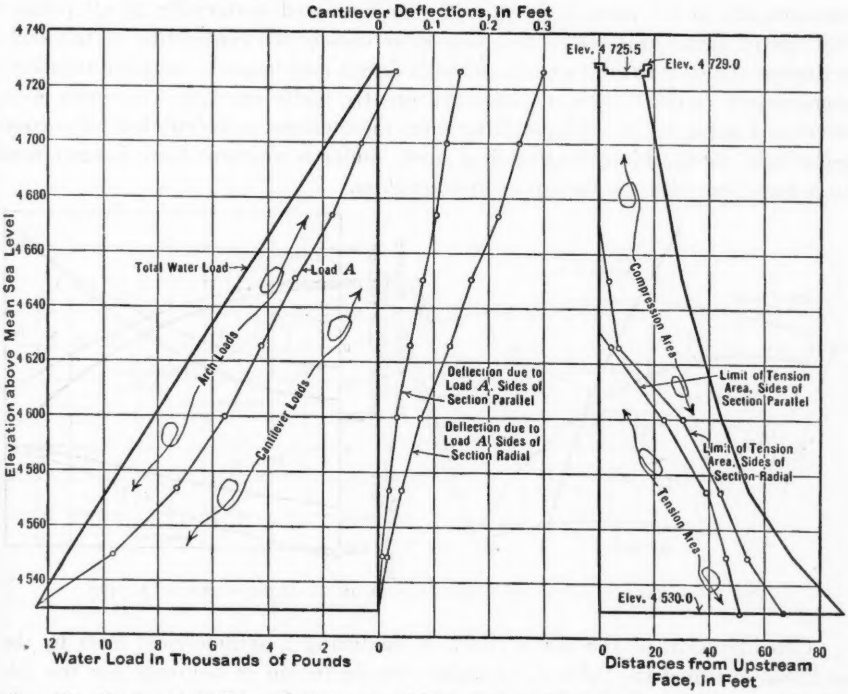


FIG. 48.—CHANGES IN TENSION AREA AND DEFLECTION OF CROWN CANTILEVER AT GIBSON DAM CAUSED BY CONSIDERING RADIAL SIDES OF CANTILEVER.

The effect of considering shear in the arch calculations was found to be practically negligible in the case of long thin arches, but to increase as the ratio of $\frac{t}{r}$ increased, becoming very important in the case of comparatively short, thick arches. For some of the thick arches analyzed, the deflections

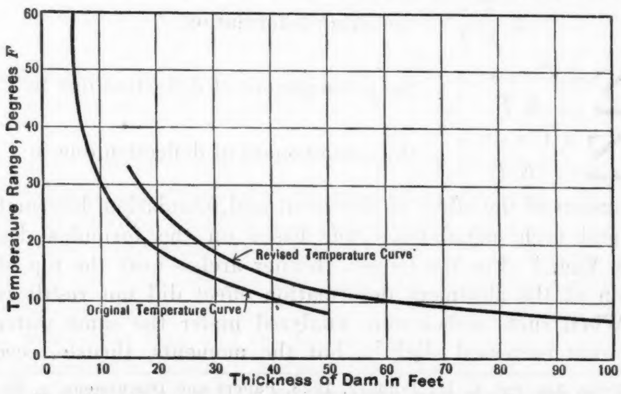


FIG. 49.—COMPARISON OF ORIGINAL AND REVISED TEMPERATURE CURVES USED IN DESIGN OF GIBSON DAM.

caused by shear were found to equal or exceed those produced by thrust and moment. Both positive and negative moments were found to be decreased numerically at all points; the thrust was increased materially at all points; the line of thrust was found to follow more closely the center line of the arch, reducing the eccentricity greatly at both crown and haunch; and the resultant compressive stresses were not changed greatly while the tensile stresses were decreased appreciably. Fig. 50 illustrates these effects as calculated for a comparatively short, thick, symmetrical arch, under a uniform load, investigated in connection with the design of another dam.

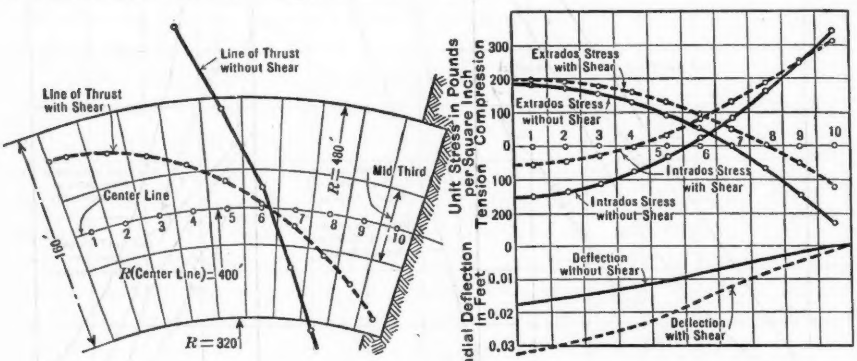


FIG. 50.—EFFECT OF CONSIDERING SHEAR IN ANALYZING ARCH ACTION.

Consideration of the shear effect in analyzing a symmetrical arch by the methods outlined in Table 5,* requires the inclusion of columns for the following quantities, the notation being the same as that in Table 5:

$$V_L = \text{ the shear due to external loads.}$$

$$\frac{3}{E}\sum \frac{S\ V_L \sin\alpha}{T} = \text{ the } x\text{-component of deflection due to } V_L.$$

$$H_c \sin\alpha = \text{ the shear component of the crown thrust.}$$

$$H_c \sin\alpha - V_L = \text{ the net shear, } V.$$

$$3\ \frac{V\ s}{E\ T} = \text{ the shear deformation.}$$

$$\sum \frac{3\ V\ a \cos\alpha}{E\ T} = \text{ the } y\text{-component of deflection due to } V.$$

$$-\sum \frac{3\ V\ s \sin\alpha}{E\ T} = \text{ the } x\text{-component of deflection due to } V.$$

The inclusion of the effect of abutment and foundation deformation in the cantilever and arch calculations was based on the formulas developed by Dr. Fredrik Vogt.† For the longer, thinner arches near the top of the dam, the inclusion of the abutment deformation effect did not result in material changes. When such arches were analyzed under the same water load the deflections were increased slightly, but the moments, thrusts, eccentricities,

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 88, *et seq.*

† *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 554.

and stresses were not appreciably altered. However, as regards comparatively short, thick arches the effects were important. For such arches the inclusion of the abutment deformation effect increased the positive moments, decreased the negative moments, decreased the thrust, increased both compressive and tensile stresses at the crown, decreased both compressive and tensile stresses at the abutment, increased the eccentricity of the line of thrust, and practically doubled the deflections. Fig. 51 shows the results of an analysis of one of the heavier arches of the 405-ft. Owyhee Dam now being built on the Owyhee Irrigation Project of Eastern Oregon, no direct comparisons being available for the arches of the Gibson Dam.

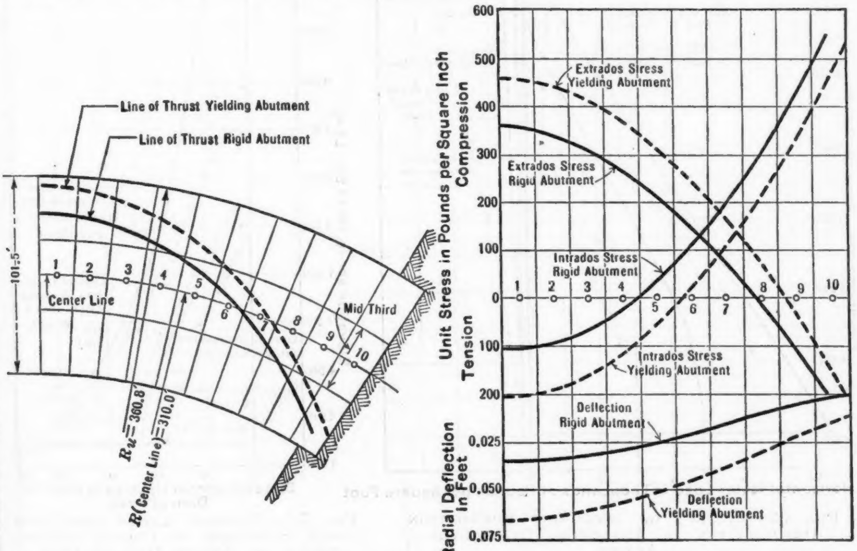


FIG. 51.—EFFECT OF CONSIDERING ABUTMENT DEFORMATION IN ANALYZING ARCH ACTION.

In the analysis of cantilevers under the same loads consideration of foundation deformations resulted in increased bending, but did not alter the stresses. Consideration of foundation movements introduced an initial shear deflection at the base, an initial twisting at the base, and resulted in increased deflections throughout the height of the dam. Fig. 52 illustrates these effects at the crown cantilever of the Gibson Dam when analyzed under the final load conditions.

Of course, the effects shown in Figs. 51 and 52 do not represent the final result of considering foundation and abutment deformations. The final effect is a new load distribution between arch and cantilever elements and an altered stress distribution. The investigations made by the U. S. Bureau of Reclamation thus far do not show whether the maximum cantilever and arch stresses will always be changed in the same direction by the inclusion of such considerations, or whether they will be changed in location, or increased in some places and decreased in others. A great many uncertainties are involved in the consideration of foundation and abutment movements, such as the value

of the modulus of elasticity for the rock formations. In the case of the Gibson Dam the modulus of elasticity for the limestone foundation and abutments was assumed to be the same as for the concrete which, on the basis of laboratory tests, was taken as 2 000 000 lb. per sq. in. As Dr. Vogt stated in the previously mentioned discussion, "no calculation of this kind may claim to be more than a rough approximation".

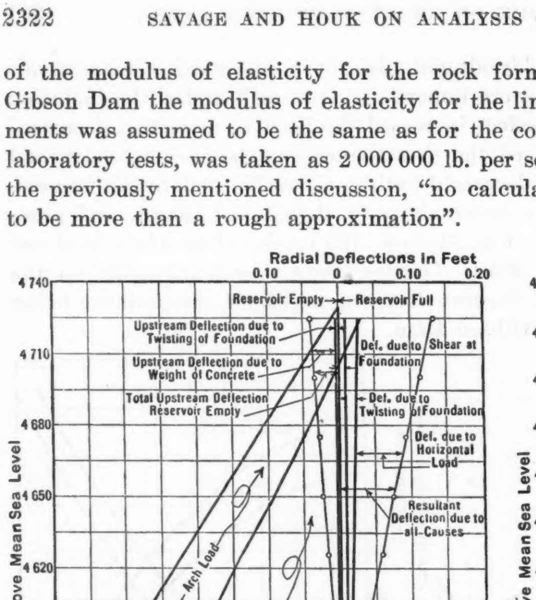


FIG. 52.—EFFECT OF INCLUDING FOUNDATION DEFORMATION IN ANALYZING CANTILEVER ACTION.

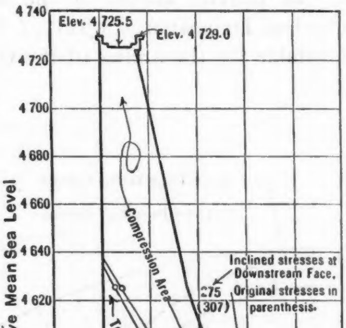


FIG. 53.—TENSION AREAS AND MAXIMUM STRESSES IN CROWN CANTILEVER OF GIBSON DAM BY ORIGINAL AND REVISED METHODS OF CALCULATION.

Assuming arch action to begin as soon as the reservoir begins to fill simply means that the arch deflections are to be measured from the up-stream cantilever deflections which occur when the reservoir is empty, instead of from a vertical axis as was done in the original calculations. Since the contraction joints are to be thoroughly grouted during the late winter, after the setting heat has been dissipated, the assumption made in the final analysis is believed to be more nearly correct than the one originally made. A number of electrical resistance thermometers are being installed in the Gibson Dam so that definite knowledge of internal temperatures will be available at the time the joints are grouted.

Figs. 53 and 54 compare the results obtained by the revised methods of analysis with those obtained by the original methods. Fig. 53 shows the differences in tension areas and maximum cantilever stresses at the crown cantilever, while Fig. 54 shows the differences in load distribution and deflections at the cantilevers, *A, C, F, G, H,* and *I*. By referring to Fig. 53 it will be noticed that for the crown cantilever the final result of all the revisions in

methods was a slight reduction in tension area and maximum stresses, the latter being reduced to stresses parallel to the down-stream face by Cain's theory in both cases. Reference to Fig. 54 shows that the deflection and load distribution curves were not altered appreciably, except in the case of the latter, at the lower elevations of the more central cantilevers where appreciable loads were transferred from the cantilever elements to the arch elements.

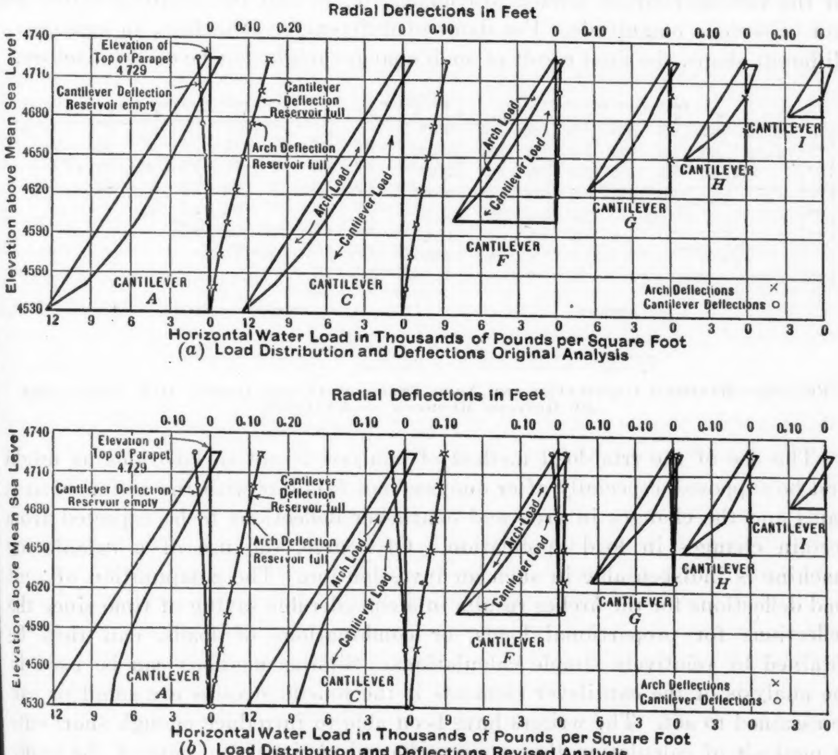


FIG 54.—COMPARISON OF LOAD DISTRIBUTION AND DEFLECTIONS OF GIBSON DAM BY ORIGINAL AND REVISED METHODS OF ANALYSIS.

Fig. 55 shows the maximum cantilever and arch stresses in the Gibson Dam as calculated by the revised methods of analysis. Similar data are available for the original investigations, but the stresses are in no cases greatly different from the final results. The several revisions introduced since the first analysis was made, decreased the maximum cantilever stress from 620 to 605 lb. per sq. in., but did not change its original location which had been found to be at Elevation 4 560 on the down-stream face of the dam, as shown in Fig. 53. They increased the maximum arch stress from 309 to 364 lb. per sq. in. and changed its location from the extrados at Elevation 4 675 at the crown to the extrados at Elevation 4 650 at the crown. Maximum arch stresses at the lower elevations were increased by greater proportional amounts, due to the change in load distribution, but the final stresses were lower than at Elevation 4 650, as shown in Fig. 55.

The results of the revised analysis of the Gibson Dam were very gratifying. They showed that the section as originally designed is satisfactory and that the maximum stresses to be expected will be well within the limits of safety. The fact that the revised analysis did not show results greatly different from the original analysis does not mean that the modifications were not worth while. It simply means that, in the case of the Gibson Dam, the effects of the various changes tended to compensate, so that the resultant effect was not of serious magnitude. For dams of different cross-section, in canyons of different shape, the final result of such changes might not be so satisfactory.

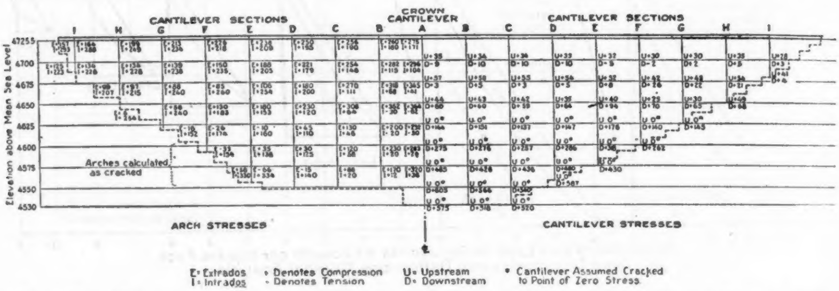


FIG. 55.—MAXIMUM CANTILEVER AND ARCH STRESSES IN THE GIBSON DAM CALCULATED BY REVISED METHODS OF ANALYSIS.

The use of the trial-load method of analysis is not so laborious as might first be supposed, especially after one becomes familiar with the action of arch dams and the changes in arch and cantilever deflections to be expected from certain changes in load application. Of course, the use of a calculating machine is indispensable in such an investigation. The computation of unit load deflections for the arches results in a considerable saving of time since the deflections for proportional loads, or combinations of loads, can then be obtained by relatively simple calculations. Similar methods can be used in the analysis of the cantilever elements if the tensile stresses are small or can be assumed to act. The writers have been able to introduce enough short-cuts in methods of calculation that an experienced engineer, in spite of the many additional considerations which have been introduced, can make a reasonably satisfactory analysis of a symmetrical dam within three weeks.

The use of the trial-load method of analysis is believed to be advisable in the case of any important dam. For the smaller dams at symmetrical sites, where the profiles across the canyon contain no decided irregularities, the use of Professor Cain's formulas is probably sufficient. The writers have found his formulas very satisfactory in determining the proper dimensions of such structures. They have also found them very useful in getting a start on the trial-load method. Their use in the case of the Gibson Dam might have been sufficient inasmuch as the load distribution curves show that the loads carried by the arches were nearly uniform, although there would have been some uncertainties involved in the application of the formulas because of the fact that the arches are not of constant thickness. However, considering the limits of knowledge of arch dams at the present time (particularly as regards the

action of the parts of the dam that are subjected to tensile stresses), it is believed that any important dam, such as the Gibson Dam, should be carefully investigated by the trial-load method, even though it may be located at a symmetrical site containing no pronounced irregularities in profile. In the future, after a large number of dams have been analyzed by the trial-load method, designers may be able to predict, from the shape of the canyon and the smoothness of the profile, whether or not an analysis by such method is necessary.

FRED A. NOETZLI,* M. AM. SOC. C. E. (by letter).†—Much effort has been spent during recent years to evolve an analysis of arch dams which should accord, as nearly as practicable, with the theoretical principles used in the design of other types of concrete structures.

The authors base their method on the generally accepted assumption that an arch dam is composed of a system of horizontal arches and vertical cantilevers. The partial loads carried by the individual elements are then determined by successive trials until the deflections of the two systems are equal. The principal advance represented by this paper is in the number of vertical sections considered, and the development of practical methods for the required computations.

The fact that this method of analysis originated in the offices of the U. S. Bureau of Reclamation and has been used in the design of several large arch dams, notably the Horse Mesa, Gibson, and Owyhee Dams, should give it much weight.

In certain respects, the same principles were used in the design of the La Jogne Arch Dam,‡ in Switzerland, although the required load distribution was determined by a different method. For that dam there were investigated nine vertical cantilevers and four horizontal arch elements. The partial loads carried by each element were evaluated by a mathematical process so that the deflections of the arch and cantilever elements were identical at corresponding points of intersection of the two systems. The tension stresses in the La Jogne Dam were limited to 52 lb. per sq. in. (4 kg. per sq. cm.), however, and secondary arches were not considered.

In detailed application the authors' method utilizes assumptions which mark a radical departure from the principles ordinarily used in the design of concrete structures in general, and dams in particular. For instance, the authors frankly assume that high tension stresses and numerous cracks are likely to occur in arch dams under load. By their method of analysis the parts of the arch and cantilever sections presumably cracked or in tension are neglected, and the final analysis of stresses is made by considering the reduced sections, that is, those in compression, only.

When, in 1920,§ the writer suggested that arch dams designed according to the cylinder formula were likely to develop cracks in both horizontal and

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† Received by the Secretary, July 2, 1928.

‡ Dr. A. Stucky, "Etude des Barrages Arques," *Bulletin Technique* de la Suisse Romande, Lausanne, 1922.

§ "Gravity and Arch Action in Curved Dams," *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

vertical planes, criticism arose. It is a source of satisfaction to the writer that his conclusions, as then stated, have since been substantially vindicated by tests.

There is some uncertainty regarding the depth of the open cracks assumed by the authors and the corresponding forces acting in the cracks due to the hydrostatic pressure of the water in the reservoir. Evidently, the cracks on the air side of the dam are not subject to such pressure.

After a crack has developed in the concrete, the adjacent parts, formerly stressed in tension, will immediately be relieved of stress by the formation of the crack. There is no reason for additional cracks forming in the immediate vicinity of the first one, and the thickness, area, and moment of inertia of the primary and secondary arches are substantially the same, except, of course, in the immediate vicinity of the cracks.

Thus, in an arch dam subjected to load there may be excessive tension stresses at both abutments at the extrados and at the crown of the intrados, producing cracks at all three points. In the immediate vicinity of the cracks, areas that were plane before bending evidently are no longer plane after bending of the arch. In other words, deformations equivalent to a slight rotation are taking place near the cracks. This has an effect similar to that of "restrained" hinges, and the "hingeless" primary arch with cracks at both abutments and at the crown, therefore, will be converted into a structural element resembling a three-hinged arch.

Secondary arching may occur in structures before cracks have occurred, in accordance with Castigliano's law of "least work". This is observed frequently in tunnel excavations. In some such cases, material has been removed to the surface of the theoretical secondary arch without impairing the strength of the structure.

It has been thoroughly demonstrated that the older and simpler methods of designing thick arch dams were at best only roughly approximate and frequently erroneous. The need of refinements in design was evident, and has led to many studies and much progress in the art. The advances in theory, however, have also served to emphasize the inherent uncertainties involved in the assumptions on which the theory must be applied. In other words, the more designers are able to visualize the mechanics of the thick arch dam, the more they appreciate its complexity and the practical limitations of its accurate stress analysis. In view of these multiplying difficulties, to say nothing of other aspects of the matter, it is perhaps indicated that future efforts should be directed toward alterations in the type of structure, such as would eliminate, in part at least, if possible, some of the uncertainties which are inherent in, and somewhat peculiar to, the thick arch dam. In other words, a too great complexity of structural action may reasonably suggest the desirability of a change in the design in the direction of simplicity.

An effort in this direction, with gratifying success, was made by the writer in the design of the Railroad Canyon Arch Dam, in California, built in 1927-28. The dam is approximately 100 ft. high. It is of the variable radius arch type and has an up-stream slope at the arch crown of 10 on 4. Instead

of designing the dam on the ordinary arch principle, then assuming cracks in the concrete, and neglecting the concrete cracked or in tension, as in the authors' method, the writer shaped the arches, as constructed, directly along the outline of the "secondary" arches, thereby eliminating tension, at least in any appreciable amount, together with the numerous attendant uncertainties. Incidentally, this design saved the concrete which, under the assumption of either "reduced" or "secondary" arches as portions of thick arches, would have been placed in the structure without any considerable increase in its strength.

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PAPERS AND DISCUSSIONS



# INLETS ON SANDY COASTS

## Discussion*

By Messrs. Morrough P. O'Brien, Elliott J. Dent, Augustus Smith, and Victor Gelineau.

Morrough P. O'Brien,† Jun. Am. Soc. C. E. (by letter).‡—In connection with this paper, it is interesting to compare the American methods of handling such problems with those of European, and especially German, hydraulic engineers. In addition to theoretical and practical considerations and field observations the latter use model studies as the basis for the design of new river and harbor works.

During the summer of 1927, the writer had the opportunity of visiting the laboratory at Wilhelmshaven, maintained by the German Navy for studying problems arising in connection with the German harbors. At that time, experiments were being made on a model (constructed to a scale of 1:250), of the approach to Bremen in the vicinity of Wangeroog Light. The model occupied an area 100 by 46 ft., and was built of sand and surfaced with a neat cement. The currents in the prototype result principally from the tides, and the purpose of the studies was to discover a method of using these tidal currents to cut and maintain a deeper channel in the sandy bottom. No attempt was made to determine, quantitatively, the erosion in the model but, instead, the velocities in the model were carefully found and used, in combination with data on erosive velocities, to compare the effectiveness of the various designs proposed. The tides in the model were produced by means of a cam operated by clockwork which controlled its supply and discharge. The waves were propagated by means of special apparatus. After the model was built, the first problem was to regulate the currents so that they corresponded to those which had been previously observed in the prototype by means of floats. After the water quantities and points of inlet and

* Discussion of the paper by Earl I. Brown, M. Am. Soc. C. E., continued from August, 1928, *Proceedings*.

† Univ. of California, Berkeley, Calif.

‡ Received by the Secretary, June 25, 1928.

discharge had been adjusted so as to give these conditions, the various proposals for improving the channel were built to the proper scale and their action was observed. The bottom velocities were studied by means of potassium permanganate crystals, and the surface velocities were measured by means of floats used in connection with a co-ordinate system of fine wires extending over the whole model. The observations were made from a traveling platform. At the time of the writer's visit, a satisfactory arrangement had been found and construction was to start immediately.

This laboratory, established in 1907, has performed sufficiently important services in the construction of the Harbors of Helgoland, Bremen, Bremerhaven, and Wilhelmshaven to warrant the construction of a new laboratory in 1919, in spite of the financial depression in Germany following the World War.

Although the study of such problems as the formation, maintenance, and migration of inlets, by means of models, probably would require rather large and expensive equipment, some of the problems mentioned in the paper, such as the laws governing the flow through a single inlet between a tidal sea and an inland basin, could be studied in the laboratories already built at some American universities. Such model experiments not only would aid practicing engineers, but also would serve to give the engineering students that judgment which is such an important part of hydraulic design. Most hydraulic textbooks treat the subject in so certain a manner that the average engineering student leaves the university without even suspecting the great uncertainty and complexity involved in such an apparently simple phenomenon as the flow of water. The usual laboratory work at American engineering schools does not give as clear a view of this difficulty as would be obtained from experiments on models, in which slight changes in construction very often bring about relatively great variations in the manner of flow. The defeat of the bill providing a National Hydraulic Laboratory precludes, at least for the present, the aid of such an institution in the solution of the larger river and harbor problems of the United States, but the establishment of smaller laboratories for such studies at the engineering schools would help to solve many problems, especially those connected with water power and irrigation. It would also raise the general level of hydraulic knowledge by sending out graduates equipped with a sounder basis of judgment.

Elliott J. Dent, M. Am. Soc. C. E.* (by letter).†—The subject of this paper is of such importance and has been treated in such a thorough manner that a discussion must, almost of necessity, be limited to a few selected details.

In his "Synopsis"‡ and "Introduction"§ the author calls attention to the importance of the effect of inlets on beach stability, and expresses the opinion that permanent stone jetties are considered the most reliable means for controlling inlets. The writer most emphatically concurs in these statements.

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† Received by the Secretary, July 9, 1928.

‡ *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 505.

§ *Loc. cit.*, p. 506.

The use of the automobile, the extensions of the highway systems, and the ability of the average citizen to spend more and more time in satisfying his recreational instincts, have increased the number of visitors to the beaches until the figures are truly astounding. Moreover, there is reason to believe that future figures will make present ones look pitifully small. No argument as to the necessity for improvement and stabilization of the beaches is necessary.

*Effect of Inlets on Beach Stability.*—In Fig. 11(*a*) the writer has attempted to show his conception of the principal sand movements in the vicinity of a normal, unimproved inlet.

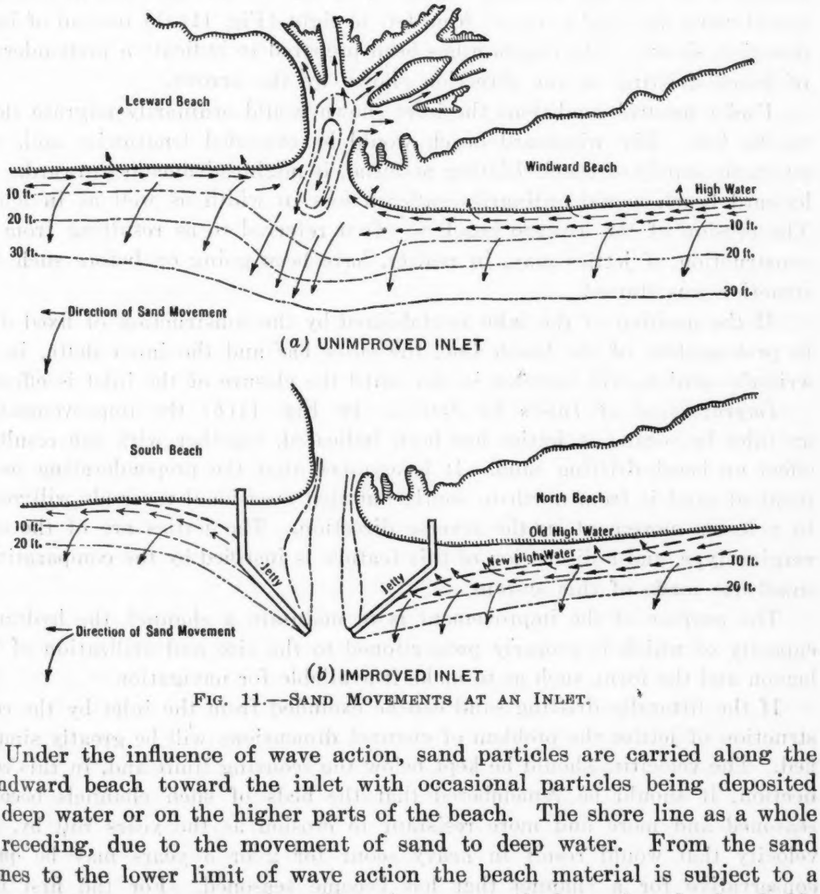



FIG. 11.—SAND MOVEMENTS AT AN INLET.

Under the influence of wave action, sand particles are carried along the windward beach toward the inlet with occasional particles being deposited in deep water or on the higher parts of the beach. The shore line as a whole is receding, due to the movement of sand to deep water. From the sand dunes to the lower limit of wave action the beach material is subject to a constant re-arrangement of profile. At any particular point the inevitable and continuous loss of beach material may be temporarily masked by a re-arrangement of the shore sands in such a manner as to increase the acreage of the dry land.*

* "The Preservation of Sandy Beaches in the Vicinity of New York City," *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1786.

In the vicinity of an inlet another major force comes into action and the hydraulic currents carry large quantities of sand to the inner delta during flood-tide periods and to deep water off the outer bar during ebb-tide hours. A shifting of the inlet and re-arrangement of the material by natural or artificial methods may eventually result in the utilization of the delta deposit, but the transfer of sand from the open beach to the delta represents a present loss of considerable magnitude, and a future small gain can only partly offset such loss. Sand moved to deep water during ebb tide is permanently lost.

Some sand may follow a zigzag course and eventually pass the inlet and become a part of the leeward beach. A change in the direction of the wind would cause the sand to travel from left to right (Fig. 11(*a*)) instead of in the direction shown. The diagram has been prepared to indicate a preponderance of beach drifting in the direction shown by the arrows.

Under natural conditions the inlet shown would ordinarily migrate slowly to the left. The windward beach would be extended lengthwise and, with an ample supply of beach-drifting material, it might advance oceanward. The leeward beach would ordinarily suffer a loss in width as well as in length. The erosion of the leeward beach so often referred to as resulting from the construction of jetties may, in reality, have been going on before such construction was started.

If the position of the inlet is stabilized by the construction of fixed dikes in prolongation of the beach line, the outer bar and the inner delta, in the writer's opinion, will increase in size until the closure of the inlet is effected.

*Improvement of Inlets by Jetties.*—In Fig. 11(*b*) the improvement of an inlet by means of jetties has been indicated, together with the resulting effect on beach-drifting sand. It is assumed that the preponderating movement of sand is from north to south, but that occasional reversals will result in a lesser movement in the reverse direction. The jetties are of the converging type, and a discussion of this feature is justified by the comparatively small use made of this system.

The purpose of the improvement is to maintain a channel, the hydraulic capacity of which is properly proportioned to the size and utilization of the lagoon and the form, such as to make it available for navigation.

If the littorally drifting sand can be excluded from the inlet by the construction of jetties the problem of channel dimensions will be greatly simplified. The velocities should be kept below the scouring limit and, in this connection, it should be remembered that the beds of such channels become seasoned and more and more resistant to erosion as the years roll by. A velocity that would result in heavy scour for 2 or 3 years may be quite conservative for a channel that has become seasoned. For the first few years, scour may be permitted, the maintenance charges being comparatively heavy. The original channel may be secured by dredging in quiet water after the jetties have been built.

In connection with scour it is also well to remember that Nature's usual process is to restore equilibrium by the excavation of deep pockets and the

construction of intermediate bars. A scouring velocity does not necessarily result in an enlargement of the hydraulic capacity of the channel, it often increases the roughness factor until the capacity is actually reduced. During the process of re-adjustment the bed is also seasoned to resist the new velocities whatever they may be.

With parallel jetties the dimensions of the channel are quite rigidly fixed in the first instance, whereas with converging jetties changes can be effected at small expense as the necessity arises.

With converging jetties a wave expands as soon as it passes the entrance and flattens out so as to become harmless. With parallel jetties the waves run through the channel for considerable distances before their force is dissipated.

Regardless of whether the jetties are converging or parallel they should extend to a depth at which the effect of wave action on the movement of sand is of small moment. If the ocean bed slopes steeply it may be necessary to carry the jetties to a depth of 20 ft., or more. If, on the other hand, the offshore slope is gentle and the 10-ft. depth at low water is 1 500 to 2 000 ft. from shore, the force of the storm waves will be dissipated in the several lines of breakers, and the littoral movement of sand may be controlled by jetties extending to a comparatively shallow depth.

Even if a comparatively shallow channel will meet the needs of hydraulic capacity and navigation, the jetties must be extended far enough to eliminate largely the littoral drift of sand into the channel. After the jetties are built the under-water contours will advance seaward, on one side, at least, and an extension of the jetties may eventually be required. The depth of channel required for navigation must also be considered in connection with fixing the minimum length of the jetty.

When the littoral drift is largely in one direction the construction of jetties will result in an accumulation of beach sand and the extension of the beach area on the side from which the drift comes (see Fig. 11(*b*)). The impounding of sand in this manner is not an unqualified benefit, but in this position it does less harm than would otherwise result from its deposit in an outer bar and inner delta. Complete stabilization of the beach lines on each side of an inlet does not seem practicable.

On the leeward side of the inlet the effect of jetty construction is often over-estimated. Under natural conditions such a beach receives little material from the windward side and the leeward beach is ordinarily washing away before any jetties are built. The fact, so often noted, that a leeward beach has been eroded subsequent to the construction of jetties, does not necessarily prove that the jetties were the cause of such erosion.

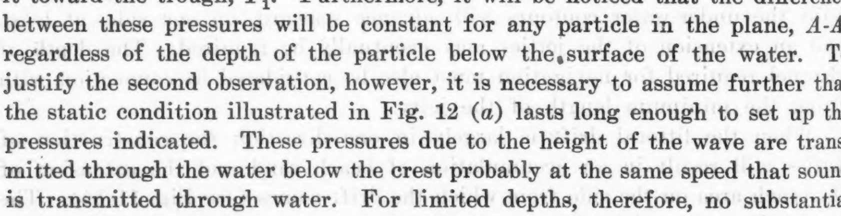
*Closure of Unnecessary Inlets.*—The author has called attention to the fact that beaches in the vicinity of inlets are normally unstable while those far away from inlets are ordinarily quite stable; that the volume of littorally drifting sand is independent of the size of the inlet; that a large volume of flow in a large inlet can better cope with this drift than the lesser flow of a small inlet; that there are many minor inlets breaking the continuity of the

beaches but they are useless for navigation and of small value as feeders of circulating water to the lagoons; and that, in certain localities, several lagoons each of which is now connected to the ocean by a minor inlet might be interconnected and served by a single larger inlet.

It would seem that a logical development of beach lines might include the closure of many inlets now open and a concentration of energies on the improvement of those selected for retention. This treatment is particularly applicable to localities where intensive use justifies large expenditures in order to secure large returns.

AUGUSTUS SMITH,* M. AM. SOC. C. E. (by letter).†—In discussing this paper, the writer has no thought of disagreeing with any of the author's observations or conclusions, but hopes that he may add some trifling detail that Colonel Brown has omitted or disregarded. To begin at the beginning of most of the phenomena alluded to by the author, namely, beach erosion, consider the fundamental nature of ocean waves and notice how the wave oscillations are converted into rushing torrents as the waves roll in over shallowing water to a sandy beach.

In Fig. 12 (*a*) suppose the undulating line, $W_1\ T_1\ W_2\ T_2$, to represent the surface of water disturbed by waves. Consider a particle of water, $P$, somewhere under the sloping side of the wave, $W_1$. It will be noticed that the pressure, $P_2$, urging this particle toward the crest of $W_1$, due to its depth, $d_2$, below the surface, is less than the pressure, $P_1$, due to the depth, $d_1$, urging it toward the trough, $T_1$. Furthermore, it will be noticed that the difference between these pressures will be constant for any particle in the plane, *A-A*, regardless of the depth of the particle below the surface of the water. To justify the second observation, however, it is necessary to assume further that the static condition illustrated in Fig. 12 (*a*) lasts long enough to set up the pressures indicated. These pressures due to the height of the wave are transmitted through the water below the crest probably at the same speed that sound is transmitted through water. For limited depths, therefore, no substantial error will be made if it is assumed that these pressures are set up instantaneously under the crest of the wave all the way from the surface to the bottom.




(*a*) DEEP WATER OR LEVEL BOTTOM (*b*) SHALLOW WATER AND SLOPING BOTTOM

FIG. 12.—OUTLINE DIAGRAM OF WAVE MOTION.

A particle of water in the plane, *B-B*, taken nearer the crest of the wave, or a particle in the plane, *C-C*, nearer the trough (in both of which cases the slope or angle of the surface of the water to a horizontal plane is less than

* Member of Board of Commerce and Nav. of the State of New Jersey, Bayonne, N. J.

† Received by the Secretary July 17, 1928.

at the plane, $A$-$A$), would likewise be urged toward the trough, $T_1$, by a force somewhat less than $P_1 - P_2$ which acted on the particle, $P$. Nevertheless, the force would be a constant for every particle in the plane, $B$-$B$, no matter how far that particle might be below the surface; and, likewise, the force acting on particles in the plane, $C$-$C$, would be a constant for every particle in that plane.

Therefore, the pressures on particles of water under a surface disturbed by waves urge the particles from positions under the crests toward positions under the troughs, and this urge is independent of the depth for the limited depths considered in this discussion.

The result is that a great volume of water under the crest of a wave, such as $W_2$, is forced in opposite directions. The volume under the slope, $W_2\ T_1$, rushes toward the trough, $T_1$, and the volume under the slope, $W_2\ T_2$, rushes toward the trough, $T_2$. Likewise, the volume of water under the slope $W_1\ T_1$, rushes toward the trough, $T_1$.

If the bottom is level, or if the water is so deep that the bottom can be considered to be level, the volume of water vertically under the slope, $W_1\ T_1$, will have exactly equal and opposite mass and momentum to the volume under the slope, $T_1\ W_2$. These masses will collide and their momentum will be absorbed by building up a new wave crest, $W_3$, shown by the dotted line, Fig. 12 ($a$), directly over the trough, $T_1$. The crest, $W_3$, will rise exactly as high as the crests, $W_1$ and $W_2$. The horizontal motions of the particles originally in the slopes considered will be stopped by the new crest, and no net resultant vertical or horizontal motion will be produced. In short, this wave motion is a perfect vibration under the influence of gravity, quite analogous to the swing of a pendulum. In natural waves this simple vibration is somewhat obscured by overlapping independent vibrations resulting in irregular heights and hollows. The principle underlying the vibration is nevertheless easily discernible. When the wind is blowing there is an additional phenomenon spread over the surface by the friction of the air which drags some water with it.

If the bottom slopes up toward a beach and the water is comparatively shallow so that the volume of water under the wave slope, $W_1\ T_1$, is appreciably less than that under the wave slope, $W_2\ T_1$, as shown in Fig. 12($b$), when these moving volumes collide the mass from under the slope, $W_2\ T_1$, will slightly overcome the mass from under the slope, $W_1\ T_1$, and there will result a net movement of some water toward the beach. The new crest, $W_3$, will represent the impact of the balanced energies, which collided and were then absorbed and will be somewhat less than the crest, $W_2$, the decrease of size of wave being a function related to the velocity imparted to the water from volume coming from under the slope, $W_2\ T_1$, toward the beach.

It has been stated that the force acting on submerged particles of water, urging displacement from under the crests toward the troughs, was independent of the depths of the particles below the surface for the limited depths under consideration. The resistance, however, that the particles encounter when they try to move, will increase very rapidly with the depth because of the mass of surrounding water that also has to be moved to make room.

It follows that water near the surface of a wave moves horizontally during the wave oscillation through a much longer amplitude of swing than a particle near the bottom, and during the vibration the upper particles acquire greater velocities because the total vibration has to be accomplished in exactly the same time whether the particle is near the surface or near the bottom.

When a wave is moving over a sloping bottom running up to a beach (Fig. 13 (*a*)), the effect of the greater velocity and momentum of the particles near the surface will be to make the face of the waves steeper toward the beach than toward the open sea, and to make the surface water set toward the beach with cumulative velocity until a hydrostatic head is built up near the beach that sets a return current from the beach toward the sea. The return current toward the sea is forced to flow near the bottom where the opposing tendency for the particles of water to move toward the beach, by wave action, is less.

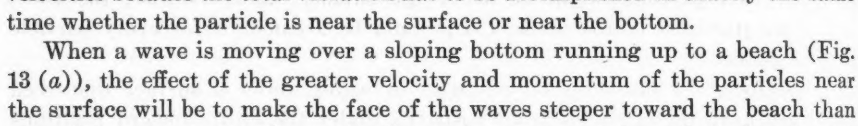


FIG 13.—OUTLINE OF WAVE MOTION AT THE BREAK.

As the water becomes shallower the face of the wave toward the beach becomes steeper and steeper, and the motion shoreward, of the surface water, becomes faster and faster until the wave "breaks". (See Point *A*, Fig. 13(*a*)). Then it suddenly decreases its height and expends its momentum in a surface rush of water toward the beach, which is offset after a sufficient hydrostatic head is built up by a strong outward or return current (Point *B*, Fig. 13 (*a*)), near the bottom.

The combined effect of this wave action and return current on a sandy beach which can be easily eroded, therefore, is to loosen the sand and mix it with the agitated water which is then drawn off along the bottom until the water becomes deep enough to reduce materially the velocity of the outshore current, at which point the sand carried in suspension will be dropped to form a bar (Point *C*, Fig. 13 (*a*)), of sand parallel to the beach.

As soon as this bar forms, the waves will break on it (see Fig. 13 (*b*)), but the crash of falling water will not hit the beach because the inshore surface of the bar slopes down into deeper water so that the falling wave crest will be received on other water between the bar and the shore. Thus, the destructive action of the waves breaking on the beach will be stayed.

The quick erosion of a beach, therefore, takes place during a rising storm, when the waves increase in height faster than the protecting bar (Point *C*, Fig. 13 (*b*)) forms, or on a rising tide when the depth of water on the protecting bar gradually increases, calling for more and more sand from the beach to break the waves. When the wind is blowing toward the shore the friction of the air drags water with it, greatly increasing this vibratory conversion phenomenon.

The opposite phenomenon of an offshore wind building up or restoring the beach, as described by the author,* might be expanded somewhat. When the

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 511.

wind blows offshore the level of the ocean is lowered considerably for the reason stated by Colonel Brown, namely, the dragging effect of the wind on the water. The outshore defensive bar thrown up by the recent onshore weather comes close to the surface, in fact, frequently appears above it. The waves, however, produce breakers (the result of the conversion of their momentum by the sloping bottom) moving shoreward against the wind as illustrated in Fig. 13 (*a*), except that the wind being against the motion of the surface water instead of with it, the resulting velocity of the top of the wave toward the beach is much less. The outer bar (or bars) now finds itself in the region of water moving shoreward and is rapidly cut down and pushed into the trench on the shore side of the bar. The waves now run over the trench filling it up level with sand from the bar and then finish the beach by heaping the sand smooth to the normal beach slope, the angle of which will depend on the size of the particles of sand and on the depth of the offshore water.

There is no perceptible undertow or seaward rush of water on the bottom when the wind blows offshore, because each wave sinks through the sand after it has run up on the beach, and the water returns seaward mostly underground, depositing the sand on the beach.

Probably the same quantity of water returns seaward from the beach underground whether the wind is onshore or offshore, but when the onshore rush of the ocean's surface is "savage" under the lashing of an onshore wind, the underground return is comparatively negligible, while the visible undertow above the bottom carries tons of sand seaward with it. When the inshore rush of the waves is "tamed" by the offshore wind, the undertow above the bottom is not sufficient to carry as much sand with it as the incoming wave carries in. The underground return current is nearly sufficient to return to the ocean all the water that laved the beach.

Another force tending to erode a beach when the wind is onshore and to restore it when the wind blows offshore, is the wind itself. When it blows onshore hard enough to raise the beach sand, the latter is rapidly borne landward to form sand dunes and to fill up bays and lagoons. Sometimes this occurs by direct deposition in the bays, and sometimes by temporary deposition behind the dunes. Later, the sand is washed down by rains into the bays or lagoons that usually exist in a more or less filled up condition behind the line of sand dunes. When the wind blows offshore all the loose sand not trapped by water in the bays, nor by grass on the dunes, is blown back to help restore the beach.

It is obvious that the net effect of wind is to move sand from the beach inland because more moves inward than goes back. It would seem, therefore, that the restoration or stabilization of a beach should involve periodical artificial replacement from inland sources of at least as much sand as has been moved in by the wind during the period. The City of New York has recently made two demonstrations of beach restoration and stabilization on a colossal scale. At Coney Island, sand was pumped in from the sea. At Rockaway Beach, sand was pumped back seaward from Jamaica Bay. The latter method would seem to be more in the line of stabilization.

As Colonel Brown has shown,* once the crash of the breakers on the beach has loosened the sand a littoral current, set up usually by the angle at which the waves struck the beach or for any of the other reasons he enumerates, will cause the sand to travel rapidly with the current. This beach sand in motion under water is the material which makes bays and barrier beaches and fills up inlets.

The derivation is always in the erosion of the beach somewhere exposed to the fury of the breakers backed by an onshore wind. As values of beach-front property have increased, beach erosion has become very alarming. The filling of inlets and channels in bays is always detrimental to navigation. How to cope with the situation has challenged the ingenuity of engineers and beach wardens for a long time, and yet none of the defensive measures thus far adopted can be considered entirely satisfactory.

Undoubtedly the most effective defense against the erosion of a sandy beach is the natural automatic building up of the offshore bar by the undertow or back-flow of the waves themselves. A somewhat similar defense has been manifested in the protection afforded to a beach by a wreck grounding broadside to the beach.

There is a patented breakwater system that aims to reduce the viciousness of the onshore breakers by pumping air to one or more pipe lines laid parallel to the shore and out some distance from it. The air is allowed to escape to form a screen or curtain of bubbles, thus providing a compressible cushion between the colliding wave masses and, in that way, interrupting the transmission of the wave momentum shoreward.

Other inventors have proposed other devices partly to neutralize in some way the repetitive blows of the water on the beach. There is much room for hope that some effective measures will be found along this general line of defense to reduce the force of the onshore waves during a storm so that the erosion will be negligible.

If erosion can be checked, the various forms of more or less expensive, and often unsightly, groynes and jetties which now have to be constructed to catch or trap beach sand to prevent its free migration up and down a beach and into inlets, can be simplified or, in some cases, perhaps be spared altogether.

Beach erosion is the source of all the trouble (exclusive of wind-carried sand). Beach erosion in one place often means beach accretion somewhere else; and beach accretion is also detrimental, for what advantage is there to the public in developing a beach front with expensive structures overlooking the water, if the water line then moves seaward to any great distance? Filling of inlets by migratory sand is undesirable and expensive to combat. From every point of view, therefore, the erosion of beaches should be checked if it can be done.

VICTOR GELINEAU,† M. AM. SOC. C. E. (by letter).‡—The author has undertaken the solution of the problem of determining the relation of dimensions of an inlet when such problem involves a lagoon of definite dimension with

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 507.

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‡ Received by the Secretary, August 1, 1928.

a specified tidal range in the ocean and in the lagoon. The fund of knowledge on this subject is far from adequate, a condition largely due to the fact that, as in no other field of engineering, is there such a lack of continuity in the studies and observations of the conditions at given inlets where improvement has been undertaken. This study is timely. Certain practices that have been advocated in textbooks and reports for many years past no longer apply with the force that once characterized these statements prior to the development of the modern hydraulic dredge. No longer is it necessary for the engineer undertaking the improvement of an inlet to rely entirely on jetties and other structures designed to induce what is termed "sluicing."

The subject is of great importance and is becoming enhancingly vital in many coastal communities. By far the greater part of the ocean frontage from Montauk Point, Long Island, southward to Southern Florida is composed of a barrier beach separated from the mainland by a lagoon system. Communities have sprung up on these beaches, and the mainland river ports adjacent to the beaches that are being rapidly developed, require adequate improvements of navigable waterways that they may have ocean coastwise communication with the other parts of the country. Usually, the inlet and bar control the available depth. As inlets close, the degree of salinity of the lagoon waters is affected to such an extent that fisheries of great economic value to the community are impaired or utterly destroyed. Again, as beach and lagoon-fronting communities grow, it is sometimes found advisable to pierce openings through the barrier beach in order to admit sufficient ocean water to reduce, within necessary limits, the pollution of the inland waterways by sewage and other wastes.

The author very ably discusses the essential points in the improvement of an inlet, namely, the establishment of proper hydraulic conditions to realize the maximum benefit from the establishment of proper hydraulic factors. There has been an unfortunate amount of loose thinking and utterance on this very point. As the author clearly indicates the function of an inlet between the ocean and a lagoon is to serve as an orifice or tube connecting two bodies of water under rapidly changing heads. The ocean and the lagoon are both to be regarded as pools with the entire inlet gorge to be considered as the connecting tube or orifice. Obviously, then, the effectiveness of the inlet is to be measured by what may be termed its weakest or most congested section.

Some attempts to open shallowing inlets have failed because they were too limited and restricted in scope. In the Manasquan Inlet, New Jersey, for example, it is confidently asserted that attempts to open the inlet and bar will always fail until the improvement of the entire gorge section is carried through to completion. The goal is to determine the most effective section of gorge for a given tidal range in the ocean and a given tidal range area and capacity of lagoon. It is to be remembered, however, that the improvement of the gorge and approaches of an inlet will modify more or less radically the tidal range in the lagoon and, consequently, affect the volume of water that will pass through the gorge at every tidal change. This,

volume of water is termed the tidal prism. For example, in dredging the bay section of Shark River in Monmouth County, New Jersey, the total superficial area of tidal water has been somewhat reduced; but the deepening of this bay (a vast percentage of which fell bare at ordinary low water) has probably increased the volume of its tidal prism, in addition to facilitating the propagation of the tidal currents and waves.

Under the caption, "Flood *vs.* Ebb Currents,"* the author gives a very good summary of conditions of the operation of the two currents. It is admitted, however, that the most important factor should be more strongly stressed, namely, that the great effectiveness of the flood current in transporting material is due to the fact that the ocean waves have first beaten the beach material into suspension. This most potent agency is virtually lacking in the stiller shoal and limited waters of the lagoon. All observers are familiar with the somewhat parallel condition which is created during the dredging operations, the material agitated by the cutters or buckets frequently being distributed from remote points and coloring the water sometimes for miles.

The author refers perhaps too briefly to the effect on inlets of large marsh or sand-bar areas, under the caption, "Division of Inlets."† Wherever possible the situation and contour of the marshes should be critically examined in undertaking the improvement of an inlet. Every bar or marsh area is the result of the operation of waves and currents during many years; hence they must not be disregarded in plotting the layout of an inlet. If the position of the inlet and its approach is not well balanced with respect to the marsh area it is highly probable that the inlet-opening project will fail, because of failure to concentrate the necessary flow. It is a fact that the position of marsh islands is frequently the most satisfactory indication of the location of old inlets which have closed, a situation which frequently gives rise to important questions of land title.

Under the heading, "Erosion",‡ the author argues from the principles of hydraulics that the utmost efficiency is to be sought by restricting the inflowing and outflowing water to a narrow deep channel instead of depending, as has been so frequently argued, on wide relatively shallow inlet entrances. This runs counter to the argument so often advanced that twin jetties necessarily cut down the tidal flow. In other words, this would indicate, as the most efficient section for the inlet gorge, practically a semi-circle. The author very properly qualifies this mathematical deduction by pointing out that the banks would not stand at the required slopes and, as a practical matter, if the improvement of navigation is the primary object sought, sufficient width for safe maneuvering of boats is required.

In tracing the relation between an inlet and a lagoon, the author very properly states§ that the size of the body of inland water is thus a factor in establishing the width and to some extent the depth in the gorge through which waters enter and leave the bay as the tide rises and falls. It is sub-

* *Proceedings,* Am. Soc. C. E., February, 1928, Papers and Discussions, p. 516.
† *Loc. cit.,* p. 522.
‡ *Loc. cit.,* p. 526.
§ *Loc. cit.,* p. 527.

mitted that this statement should be somewhat extended by taking into account the volume or tidal prism, since the improvement of the inlet gorge may modify more or less seriously the tidal range and tidal levels in the bay or lagoon. It is necessary to estimate the final tidal range in the lagoon.

Under the caption, "Formulas Applicable to Flow Through Inlets",* the author, first working out the slope in the inlet gorge, develops a general formula for the maximum mean velocity of water flowing through the inlet and then formulas for the volume of tidal discharge that is capable of passing through a given inlet under given conditions in the bay. Then he determines the size of bay required to allow the tide in it to rise to a given height when flowing through an inlet of given size; and, finally, the height to which the tide will rise in the bay. How well these formulas would apply in practice is a matter on which there may be some question, but the test of the formulas as applied to Absecon Inlet† is worthy of study. In discussing the limitation of the formulas, the author emphasizes the fact that the value of $C$ is seldom uniform. This is one outstanding fact that has rendered so difficult the derivation of formulas that will be applicable within relatively large limits. That is, the great difficulty has always been to determine first how far to accept the observations derived in the study of one inlet in planning works for another inlet where conditions are not closely similar; and few of them are closely similar. For example, in addition to the factors taken into account by the author, such as tidal prism, range of tide, etc., the rapidity with which a bar may be eroded, may be of considerable importance since frequently a bar or marsh inland acts powerfully in directing the flow of tidal currents. Extensive dredging operations or reclamation projects will necessarily affect the factors on which the calculations are based, although it is the duty of the U. S. War Department and State agencies to prevent operations so radical as to threaten the maintenance of navigable waterways. The author states‡ very truly that observations seem to confirm a view that inlets having a channel large enough to give a considerable value to $h$ are generally in better shape than those where the tidal variation is not felt in the bay; that is, when $h$ is at or near zero, although the latter gives the greater values for velocity. The fact is that if the tidal variation in the bay is very slight compared with the ocean tidal range, this is an indication that the inlet gorge is too restricted for the area of the bay (more accurately, the tidal volume), for the tidal variation will depend on the gorge section. The continuity of a satisfactory section of gorge must extend throughout the gorge and include suitable approaches both in the ocean and bay. Every shoal in the inlet or approach operates as a submerged weir or sill, and by reducing the efficacy of the gorge reduces the tidal variation in the lagoon. Normally, the mean level of the water in the lagoon approximates mean sea level, although this is always modified by the influence of river outflow. The approaches to the gorge are just as important as the main gorge itself in preparing plans for

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 528.
† *Loc. cit.*, p. 535.
‡ *Loc. cit.*, p. 536.

an inlet project. This point is mentioned by the author under the caption, "Friction and Other Resistances",* and specifically with reference to Manasquan Inlet.

Much sound reasoning on the problem of inlet improvement and maintenance is presented under the caption, "Preservation of Tidal Flow".† It is essential to prevent the dispersion of energy that results from the flow of water following a number of relatively unimportant channels and to secure the maximum results by suitable concentration of flow. The point is well taken that in many situations the area and tidal volume tributary to an inlet may be greatly increased by artificially creating a new point at which the tides from two inlets meet. This point of meeting is sometimes described as a bulkhead. It is usually indicated by a relatively large shallow bay or by a relatively obstructive formation of marsh islands, such as the Cedar Bonnet Islands which separate Barnegat Bay from Little Egg Harbor in New Jersey. For example, in numerous situations in the lagoon area extending along the New Jersey Coast from Bayhead to Cape May, it would be a relatively simple matter to extend the tributary tidal volume in the interest of almost any one of the inlets. Obviously, the result will be to impair the chances for maintaining the inlet, the tributary area of which is thus artificially restricted.

Under the heading, "Improvement by Dredging"‡ the point is properly stressed that dredging alone on the outer bar cannot afford a permanent improvement in depth unless the channel and inside basin are such as to promote an increased velocity so as to obtain the outer depth. In other words, this dredging, in order to be successful, must conform to the configuration of the neighboring shores and channels so that the general improvement in the hydraulic conditions is obtained. Usually, concentration of the flow must be obtained by jetties, although the supplementary dredging must be also performed as an initial operation and usually at stated periods as maintenance.

The principal functions of a jetty have been accurately stated by the author.§ On the subject of high and low jetties to which much attention has been given by many writers, the author‖ calls attention to the outstanding weakness of low jetties, which is the fact that they do not arrest the littoral drift which passes over them on the windward side and then requires removal by dredging. Obviously, much depends on whether the primary object is to arrest littoral drift or merely to guide the channel. It has been argued that the low jetties possess the favorable feature of reducing to a minimum the interference with the incoming flood tide, but as the author has expressly stated, favorable hydraulic sections of the inlet are often more important than an unnecessarily great width of the entrance. From this standpoint one of the strongest arguments offered by the low-jetty and single-jetty advocates is disputed.

The author's attempt to rationalize the design of inlet improvement structures on sandy coasts is laudable. Certainly engineers everywhere will wel-

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 536.

† *Loc. cit.*, p. 543.

‡ *Loc. cit.*, p. 548.

§ *Loc. cit.*, p. 549.

‖ *Loc. cit.*, p. 551.

come every addition to the acquisition of knowledge on this difficult subject on which so much has been written and in which so much disappointment has been encountered. Disappointment is sure to be realized if the lagoon body is inadequate to provide storage for an adequate flow at ebb and flood tides to preserve an inlet through the beach. On the other hand a vast area of lagoon does not of itself provide assurance of a deep inlet, but it does present the possibility for undertaking inlet improvement with confidence in the ultimate success of the project. The essential requirement under these conditions is then to provide an adequate channel through the gorge section and, consequently, far enough into the lagoon to attain favorable hydraulic conditions. Another vital requirement is the concentration of the flow by jetties which will also serve as barriers to arrest the littoral drift which in the last analysis in so many inlet operations constitutes the chief cause of shoaling. Dredging will almost invariably be required where shoaling has once taken place. If the dredging is done to conform to the bars and land formations which act to mould and reflect the flood and ebb channels the dredging alone may be the primary factor in attaining a solution; but normally the jetties will be required to preclude dispersion of the channels through a number of mutually destructive minor passes.

In relation to the derivation of $h$ in "Formulas Applicable to Flow Through Inlets",* the writer would suggest that the solution obtained be carefully checked by comparison with a somewhat similar lagoon on the same coast. For example, fair comparison can be made on the Jersey coast between the lagoon areas which are tributary to New Inlet, Absecon Inlet, and Great Egg Harbor. More and more as the inlets are improved with a given lagoon area, the value of $h$, the mean tidal variation in the lagoon, will approach that of $H$, the mean tidal variation in the sea. A short series of simultaneous readings at tide gauges suitably chosen will yield the information as to lag between the tides and the slope of the tidal planes proceeding inland in the basin.

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 531.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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# PAPERS AND DISCUSSIONS



## UPWARD PRESSURES UNDER DAMS: EXPERIMENTS BY THE UNITED STATES BUREAU OF RECLAMATION

### Discussion*

By Messrs. Ivan E. Houk, C. H. Howell, P. Wilhelm Werner, and H. de B. Parsons.

Ivan E. Houk,† M. Am. Soc. C. E. (by letter).‡—Some additional measurements and studies of upward pressures under the American Falls and Willwood Dams have been made since this paper was completed. In view of the fact that one or two additional points of interest are exemplified, and also because of the fact that it is desirable to furnish as complete data as possible for the use of interested engineers, it seems advisable to present the results of the more recent investigations in this discussion.

Table 6 gives the results of measurements made at the American Falls Dam since October, 1927, the arrangement of the data being the same as in Table 4.§ It will be noticed that observations have been made about once a month during the last year (1927-28). Present plans are to continue the readings at about the same interval for at least another year, and then at somewhat greater intervals for some time in the future. By such readings it is hoped to observe any increase or decrease in uplift pressures which may occur as time progresses. Thus far, no general conclusions as regards a time effect seem to be warranted.

Any thorough study of uplift pressure variations resulting from elapse of time should be based either on observations taken with the same reservoir and tail-water surface elevations, or on observations properly corrected for variations in total head. Variations in the total head at the American Falls Dam are essentially variations in reservoir levels, inasmuch as the tail-water elevation is kept practically constant in connection with the operation of the plant

* Discussion of the paper by Julian Hinds, M. Am. Soc. C. E., continued from August, 1928, *Proceedings*.

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‡ Received by the Secretary, June 13, 1928.

§ *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 704.

TABLE 6.—MEASURED UPLIFT ON BASE OF AMERICAN FALLS DAM SINCE OCTOBER, 1927.*

(All elevations are between 4 280 and 4 360 ft., see Fig. 19.†)

| Date of observation. | Reservoir water surface elevation, in feet. | Tail-water elevation, in feet. | UPLIFT PRESSURES EXPRESSED AS EQUIVALENT WATER SURFACE ELEVATION, IN FEET. | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Station 9 + 00. | | | | Station 11 + 10. | | | | Station 13 + 00. | | | |
| | | | *A.* | *B.* | *C.* | *D.* | *A.* | *B.* | *C.* | *D.* | *A* | *B* | *C.* | *D.* |
| November 21, 1927............ | 54.43 | 88.6 | 96.44 | 93.04 | 91.84 | 91.24 | 09.14 | 97.26 | 97.17 | 90.89 | 17.15 | 07.64 | 94.25 | 97.26 |
| December 30, 1927............ | 50.90 | 96.50 | 99.43 | 97.42 | 97.14 | 96.44‡ | 14.14 | 98.15 | 97.17 | 96.29‡ | 19.15 | 07.14 | 94.30‡ | 96.56 |
| January 19, 1928............ | 50.50 | 96.50 | 98.73 | 97.22 | 97.24 | 96.64 | 15 14 | 99.15 | 98.15 | 96.69 | 22.15 | 07.64 | 94.45‡ | 97.16 |
| February 18, 1928............ | 49.05 | 96.50 | 99.23 | 97.22 | 97.23 | 95.34‡ | 14.14 | 00.15 | 98.15 | ..... | 23.15 | 08.14 | ..... | ..... |
| March 19, 1928............ | 47.48 | 95.30 | 99.48 | ..... | ..... | 96.84 | 13.64 | 00.15 | ..... | 96.79 | 22.15 | 07.64 | 93.95‡ | 97.16 |
| April 19, 1928............ | 46.78 | 96.50 | 99.73 | ..... | ..... | 94.84‡ | 11.14 | 00.15 | 99.15 | 96.29 | 22.15 | 07.14 | 96.95 | 97.16 |
| May 18, 1928............ | 48.81 | 95.70 | 98.23 | 97.24 | 96.14 | 95.34‡ | 07.64 | 97.65 | 97.17 | 94.89‡ | 21.15 | 06.64 | 94.25‡ | 97.16 |

* See Table 4 for results of earlier measurements.

† *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 707.

‡ Below tail-water elevation.

of the Idaho Power Company about 600 ft. down stream from the dam. Keeping this in mind a cursory inspection of Tables 4 and 6 would indicate that for the same total head the elapse of time at the American Falls Dam has resulted in increased pressures at some of the pipes and in decreased pressures at others.

Where uplift pressures are affected by both reservoir and tail-water levels, or, in other words, where the bottoms of the pipes are connected with percolation passageways leading to both up-stream and down-stream edges of the base, the increases in pressure may be caused by either a loosening of the passageways leading through the grouted rock to the up-stream edge of the base, or by a tightening of those leading to the down-stream edge of the apron or to the open drains. Likewise, decreases in uplift pressure may be caused by a tightening of the passageways above the wells or by a loosening of the ones below. Probably both actions have been occurring to some extent at the American Falls Dam. Approximate measurements of total flow of the drains have been made at times of observing uplift pressures, but the results are somewhat erratic and show no definite increase or decrease in drainage since the reservoir was first filled. Tightening of the percolation passageways may be caused by deposition of salts from the percolating water. Such an action is believed to be responsible for a reduction in drainage flow which has been observed during recent years at the Elephant Butte Dam on the Rio Grande Project of New Mexico and Texas.

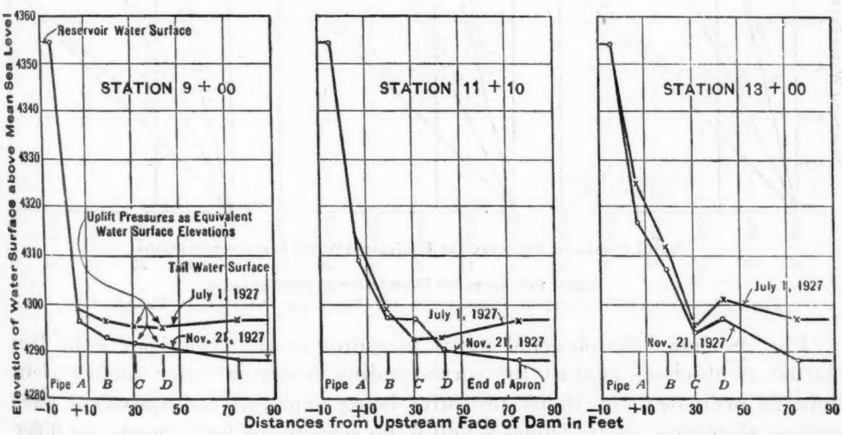


FIG. 27.—EFFECT OF VARIATION IN TAIL-WATER ELEVATION ON UPWARD PRESSURES UNDER AMERICAN FALLS DAM.

The series of uplift pressure observations at the American Falls Dam on November 21, 1927 (the first set of observations recorded in Table 6), is of special interest because of the relatively low tail-water elevation (4 286.6) which existed at that time. Such a stage is approximately 10 ft. lower than the elevation usually maintained. At the time of the November measurements the tail-water had been lowered for several days because of some gate repairs required by the Power Company. Fig. 27 shows the readings of November 21, 1927, platted with those of July 1, 1927, when the reservoir water surface was

at almost exactly the same elevation, but when the tail-water level was at Elevation 4 297.0; that is, 10.4 ft. higher than on November 21. Water surface elevations are platted as ordinates, and distances from the up-stream face of the dam as abscissas—the down-stream direction being taken as positive. The diagrams show that the uplift pressures on November 21 were considerably lower than on July 1, the differences being from 3 to 4 ft. at several of the pipes and as much as 8.0 ft. at Pipe *A*, Station 13 + 00. (See Fig. 16.*) The diagrams in Fig. 27, together with the data in Tables 4 and 6, would indicate that Pipes *C*, Stations 11 + 10 and 13 + 00, are relatively tight as regards connections with either reservoir or tail-water conditions, as stated by the author.† No explanation can be offered for the sudden increase in pressure at Pipe *C*, Station 11 + 10, between July 21 and September 19, 1927. The other pipes, some of which have at times shown water elevations slightly lower than the tail-water levels, apparently are affected to some extent at least by both reservoir and tail-water conditions.

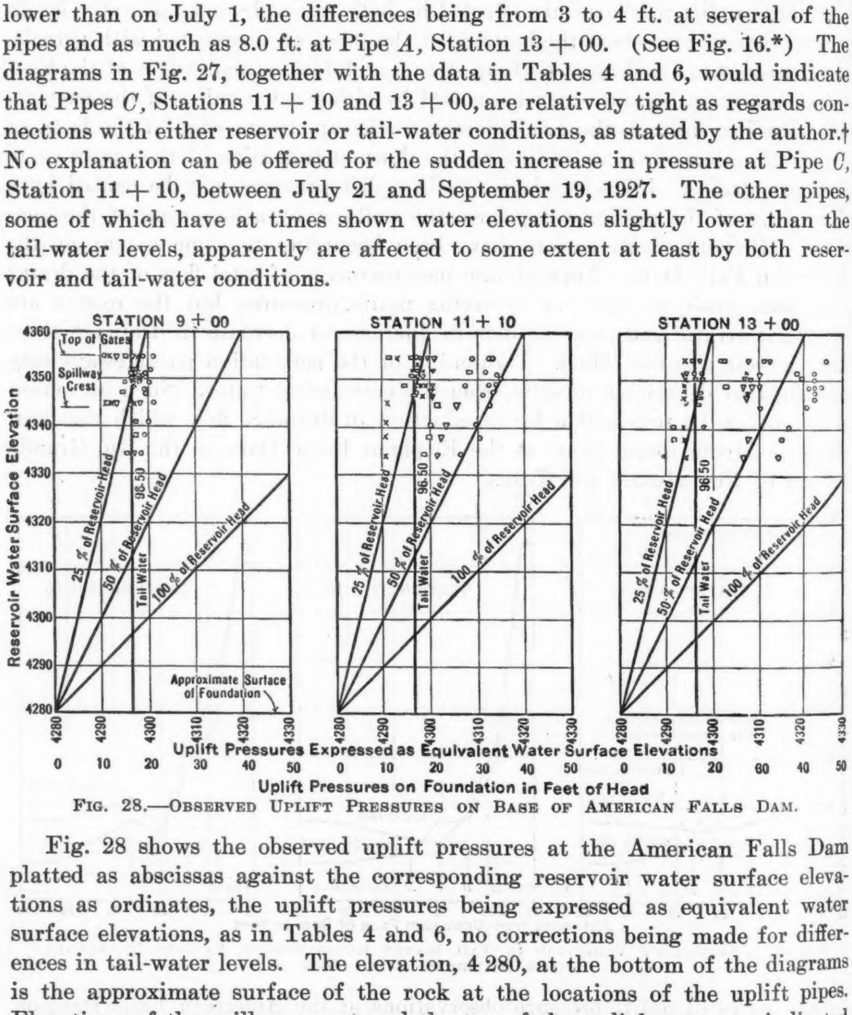


FIG. 28.—OBSERVED UPLIFT PRESSURES ON BASE OF AMERICAN FALLS DAM.

Fig. 28 shows the observed uplift pressures at the American Falls Dam platted as abscissas against the corresponding reservoir water surface elevations as ordinates, the uplift pressures being expressed as equivalent water surface elevations, as in Tables 4 and 6, no corrections being made for differences in tail-water levels. The elevation, 4 280, at the bottom of the diagrams is the approximate surface of the rock at the locations of the uplift pipes. Elevations of the spillway crest and the top of the radial gates are indicated on the diagrams by short horizontal lines, while the normal tail-water elevation, 4 296.5, is indicated by vertical lines. Diagonal lines represent uplift pressures of 25, 50, and 100% of the reservoir pressures at the up-stream edge of the base.

While the observations shown in Fig. 28 are naturally somewhat erratic, the diagrams indicate, in a general way, the variation of uplift pressures with

* *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 703.
† *Loc. cit.*, p. 704.

changes in elevation of reservoir water surface. It will be noticed that in most cases the uplift pressures fall between the 25 and 50% lines. Pressures at Pipe *A*, Station 13 + 00, are practically the only ones that fall definitely above the 50% line. These pressures fall on a diagonal line representing about 60% of full reservoir water pressure. Uplift pressures observed at Pipe *B*, Station 11 + 10, seemed to increase gradually as the reservoir filled until May 17, 1927, then to drop back about 6½ ft. by July 1, although the reservoir water surface continued to rise, reading an elevation of 4 354.46 by July 1—a total of 9.7 ft. higher than on May 17. Since July 1, 1927, the uplift pressure at this pipe seems to have remained almost constant, thus indicating that the foundation rock surrounding the bottom of the pipe has tightened appreciably. Observations at Pipe *A*, Station 9 + 00, seem to show a somewhat similar condition.

Measurements of uplift pressure at the Willwood Dam have not been made periodically for some time. In fact, only one set of observations has been secured since the paper was finished. This was on March 31, 1928, when the reservoir water surface elevation was 4 500.55 and the tail-water elevation, 4 458.50. Uplift pressures at the various pipes on March 31, expressed as equivalent water surface elevations, are given in Table 7.

TABLE 7.—UPLIFT PRESSURES EXPRESSED AS EQUIVALENT WATER SURFACE ELEVATIONS.

| Pipe. | Uplift, water surface. | Pipe. | Uplift, water surface. |
|---|---|---|---|
| *A*-1 | 4 467.2 | *C*-1 | 4 461.3 |
| *A*-2 | 4 458.4* | *C*-2 | 4 462.8 |
| *A*-3 | 4 468.8 | *C*-3 | Plugged |
| *B*-1 | 4 452.9* | *D*-1 | 4 456.6* |
| *B*-2 | 4 457.7* | *D*-2 | 4 458.7 |
| *B*-3 | 4 464.0 | *D*-3 | 4 459.1 |

* Elevations below tail-water surface are indicated by asterisks.

A comparison of the results in Table 7 with the observations of June 16, 1923, July 6, 1923, and July 1, 1926, recorded in Table 3,* when the reservoir and tail-water surfaces were not greatly different from those existing on March 31, 1928, would indicate that the uplift pressures have decreased somewhat during the five years since the dam was completed. No data are available to indicate whether this effect has resulted from a tightening of the percolation passageways above the pipes, from a loosening of the drainage passageways below, or from both.

It may be interesting to discuss briefly the methods of installing the uplift pressure pipes at the American Falls and Willwood Dams. At the latter the pipes ended at the line of contact between the rock and the concrete. The pipes were placed in holes drilled through the concrete after the body of the dam had been poured to Elevation 4 456.2, approximately 5 ft. above the surface of the rock foundation. It is presumed that the drill holes did not extend more than a few inches into the rock inasmuch as no mention of the matter

* *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 698.

is made in the installation records. At the American Falls Dam the pipes were placed before concreting, leaving the lower ends 6 in. above the rock, as shown in Section *E-E* of Fig. 16. After the concrete was poured to a height of a few feet above bed-rock, holes were drilled through the concrete at the lower ends of the pipes and approximately 5 ft. into the foundation.

Whether or not holes should be drilled from the ends of the pipes into the foundation is a debatable question. If they should extend into the rock, how far should they go? If there are horizontal seams, or percolation passageways, in the foundation within relatively short distances of the base of the dam, the uplift pressures in such places may exert some uplift on the concrete, since the weight of the intervening rock would probably be considerably less than the uplift. Drilling into the rock may intercept such areas, or passageways, and result in the measurement of uplift pressures where otherwise no uplift would be recorded. At the Gibson Dam, which is a concrete arch type, 200 ft. high, now being built by the U. S. Bureau of Reclamation on the Sun River Project of Montana, the uplift pipes are being installed in the same manner as at the American Falls Dam, but the holes at the ends of the pipes are being drilled approximately 18 in. into the rock instead of 5 ft. The depth in the foundation to which such holes should be drilled, or whether they should be drilled at all, is a problem which is not susceptible of exact solution. It must be decided on the basis of engineering judgment. Of course, the pipes might be set directly on the rock, and the ends surrounded by gravel before concreting. However, such methods would tend to intercept uplift conditions from horizontal areas considerably larger than the areas of the pipes. In most cases the writer would favor drilling into the rock about 18 in., as is being done at the Gibson Dam. Probably the depth of drilling should be decreased for comparatively low dams and increased for comparatively high dams.

As regards the proportionate area of the base on which the uplift pressures should be considered to act when designing a dam, the writer is inclined to favor using the entire area, especially if the uplift pressure curve is based on actual measurements instead of on arbitrary assumptions. Thus far, no practicable method of measuring the areas on the base of a dam on which uplift pressures are actually exerted, has been developed. Naturally, the extent of such areas will depend on the character of the foundation, the thoroughness of the grouting operations, and the efficiency of the drains. If the dam is built on a solid rock foundation, the rock thoroughly grouted along the up-stream edge of the base, and an adequate system of drains installed, the writer would expect large areas of the base to be entirely free from uplift pressure. The existence of such areas at the Willwood and American Falls Dams is indicated by the zero uplift pressures observed at several of the pipes. However, if these pressures are taken into account in determining an average uplift pressure curve, no further consideration of such areas is necessary. As far as such areas are concerned the average pressure curve should be applied to the full area of the base. If estimates of the actual areas subjected to uplift pressure are to be made and used in calculating the total uplift force, the observations which indicate zero pressure should be omitted from the diagrams used in determining the intensity of the uplift pressure.

Of course, there remains the question as to what proportion of the rock surface is in contact with the concrete in the areas where uplift pressure is observed; or, more exactly, what proportion of the areas where uplift pressure is observed should be considered as capable of transmitting uplift pressure to the base of the dam? Here, again, the writer would favor using the entire area. If a crack has formed between the foundation and the dam, as was the case at the up-stream side of the Stevenson Creek Test Dam, the full area would be effective as far as the crack extended. If the rock is of a stratified nature and a horizontal seam exists a short distance below the base, uplift pressure may be transmitted through the rock layer to the full area of the concrete even if a large proportion of the rock surface is in actual contact with the concrete. If the dam has been built on a sandstone foundation and uplift pressure exists in the sandstone, the pressure may be effective over nearly the full area of the base even if a large proportion of the sand grains are in actual contact with the concrete. In such a case the pressures would be transmitted through the sand grains from pore spaces below. In this connection it is interesting to note that H. de B. Parsons, M. Am. Soc. C. E., as the results of a large number of experiments made under varying conditions of head, depth of sand, etc., concluded* that:

"* * * it would not be safe to estimate an uplift in pervious soil of less than full pressure head acting on the total area of the base, when there is no flow of water through the soil; nor less than the full pressure head indicated by the hydraulic gradient when there is flow through the soil."

The writer's idea of the most practicable method of investigating upward pressure under dams would be to install a large number of measuring pipes, uniformly distributed over the entire area of the base. He would then plat the pressure measurements, including the observations of zero pressure, and would apply the resulting intensity curves to the full area. Such a method would make proper allowances for the areas in which no uplift pressure exists. As regards the areas subjected to uplift pressure it would be slightly on the safe side. The pressure intensities would be accurately measured, but the effective area would be assumed as equal to the maximum possible. However, the writer believes that the question of effective area where there is uplift pressure is not so important as the question of what areas are entirely free from uplift. In other words, he believes that it is important at this time to secure a large number of uplift pressure measurements uniformly distributed over the base of the dam. When such investigations have been made at a large number of dams, built on different types of foundation, the designing engineer can make a much better estimate of the uplift to be expected at a given site than is possible at present.

C. H. Howell,† M. Am. Soc. C. E. (by letter).‡—This paper gives additional information on the intensity of uplift under certain existing dams. The effect of uplift is due to two factors: One of which is the intensity of the

* "Hydrostatic Uplift in Pervious Soils," by H. de B. Parsons, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 955.

† Designing Engr., The J. G. White Eng. Corporation, City of Mexico, Mexico.

‡ Received by the Secretary, July 9, 1928.

pressure; and the other, the area of the base on which the pressure is applied. As the author states* the observations recorded do not prove or disprove the theory and assumptions ordinarily used in considering the problem. The fact that dams that have been investigated continue to stand, however, proves that the assumptions made in their designs were adequate.

The writer knows of no experiments or investigations as to the area subjected to uplift on rock foundations. This factor is just as important as that of the intensity. Its exact determination would be difficult if not impossible for the whole of any given dam site. Even if exact measurements could be obtained for one site the results would not necessarily be applicable to any other.

Water may develop pressure on the bottom area of a dam in one or both of two ways. One of these is through a faulty contact between the concrete and the top of the rock. The area thus affected must always be less than the total area as shown in Fig. 29($a$). This is true, because otherwise there would be no contact between the concrete and the foundation, which would be an impossibility. The section, $C\,L$, is the part of the unit length, $L$, of the base receiving uplift from faulty contact. The factor, $C$, may vary from zero to anything less than 1.

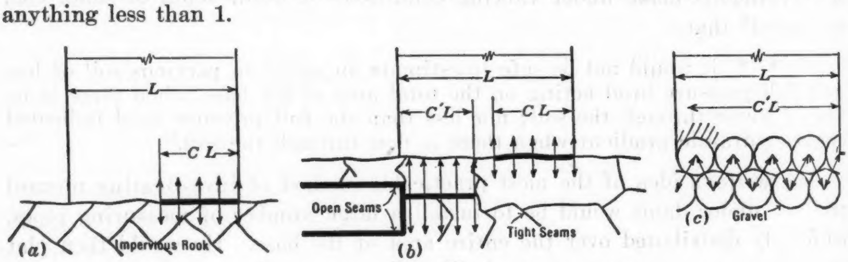


FIG. 29.

The other way for uplift to be developed is through open seams in the rock below the base of the dam (see Fig. 29($b$)). The area thus affected may easily be 100% of any given unit of area; that is, the factor, $C'$ (in Fig. 29($b$)) may vary from 0 to 1.0. Likewise, $C'\,L$ is the section of unit length, $L$, of the base receiving uplift through open seams. It seems to the writer, however, that it is unreasonably conservative to assume 100% over the entire base of the whole dam.

Fig. 29($a$) and ($b$) also illustrates the difficulty of making a quantitative determination of this part of the problem. In addition to the separate action of these two conditions the uplifting force may be the result of a combination of the two. The area subject to uplift on gravelly foundations has been shown by the author† to be approximately 100 per cent. Fig. 29($c$) shows this condition diagrammatically. In this case, $C''$ may vary from 0.9 to 1.0.

Until the areas affected by uplift can be determined the treatment of the problem on rock or similar foundations will continue to be a matter of judgment rather than mathematics. The multiplication of intensity observations alone will not serve to solve it.

* *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 685.

† *Loc. cit.*, April, 1928, Papers and Discussions, p. 941.

P. Wilhelm Werner,* Assoc. M. Am. Soc. C. E. (by letter).†—The measurements at the Colorado River Dam, although indefinite, are of a very interesting nature. The author attributes most of the irregular character of the results obtained to a variable seasoning of the filter surface. For this special case, the explanation may be exactly true, or, at least, nearly so. The writer believes, however, that in many cases the "time effect" accounts for the observed irregularities in the pressure line to an appreciable extent. A certain retardation will occur in the formation of the state of equilibrium corresponding to a new head or differential pressure.

In general, the time effect is, evidently, a condition that is rather favorable to the stability of a dam, and the writer is of the opinion that it often adds materially to the safety of the structure. This is especially true if the flood peaks are of comparatively short duration and the "retardation coefficient" of the foundation is high. The time effect is of a universal nature, and applies to soils as well as rock strata where the water is creeping in very narrow channels. Sufficient data are not yet available, however, to accomplish a general study of its character and magnitude for different materials. It is to be hoped that the problem will soon be taken up in full and given the attention it deserves among engineers.

The readings at the Willwood Dam show that uplift under dams on rock foundation is a really serious matter and that extreme care must be exercised in selecting the dam site with regard to the actual geological and hydrogeological formations. If the shale and the sandstone in this case were located in the reverse order, there would, probably, have been a considerably more unfavorable condition of uplift under certain parts of the dam. When a dam is to be built on a rock foundation composed of very different strata, there is every reason to test the foundation hydraulically in order to prevent any unexpected eventuality.

For several reasons it would be of great value to have these experiments made over a rather long period of time. It would be especially interesting to know how the uplift pressure varies with the distance to the vertical construction and contraction joints. The results at the Willwood Dam seem to give some indication of this, but sufficient data are not available to justify a conclusion.

The writer believes that a vertical contraction joint, at least when it opens up, acts somewhat as a drainage well; thus decreasing the uplift under the base of the dam. The necessary requirement is, of course, that the joint shall not become clogged and that the seepage can freely issue into the tail-water. As contraction joints in solid gravity dams are usually constructed, however, it seems rather doubtful whether it is safe to assume an effective drainage, although it is more than probable that in many cases such joints have had an appreciable effect on the stability of dams. The writer is of the opinion that this question involves an economic factor of importance, and, therefore (assuming that there are favorable conditions at the dam site), he would suggest a solid gravity dam as shown in Fig. 30.

* Stockholm, Sweden.

† Received by the Secretary, July 27, 1928.

This dam would be built in sections of ordinary length. The blocks would be separated by permanent contraction joints, extending down to rock, which would be made wide enough to permit the removal of the forms. This arrangement would cause an increased expense for shutterings, but would mean a considerable saving in concrete due to the decrease in uplift. Preliminary estimates indicate that for dams of ordinary size a saving of at least 5% of the total cost is quite feasible even under the assumption that the value of the ratio between the unit prices of shutterings and concrete is unfavorable.


FIG. 30.

The writer is well aware of the fact that, in certain respects, this suggestion is somewhat against present common practice, and a discussion would probably involve many of the most important questions relating to the design of solid gravity dams. Some of these points are in distinct favor of the new design; that is, the possibility of a convenient inspection of the joints and the facilitated dissipation of the chemical heat in the masonry. Other points seem to be more or less a matter of opinion; for example, the hypothetical mutual support between different parts of a solid gravity dam as a means of preventing ultimate failure. Whatever the judgment may be, the writer thinks the idea is worth considering.

H. DE B. PARSONS,* M. AM. SOC. C. E. (by letter).†—As knowledge regarding hydrostatic uplift pressure under dams is meager, all records of actual pressures are welcome. The profession is indebted to the U. S. Bureau of Reclamation for making the experiments reported by the author.

There is an upward pressure on the base of every structure founded below water level on fissured or pervious material. The intensity of this pressure undoubtedly would vary with differences in foundation material—its granular structure, grading of particles, fineness or coarseness, compactness, solidity, etc.

* Cons. Engr.; Prof. Emeritus, Rensselaer Polytechnic Inst., New York, N. Y.

† Received by the Secretary, August 1, 1928.

Because all the factors involved are not appraised, it is to be expected that measurements of upward pressures will often appear inconsistent, that is, will apparently lack proper relationship. The water column in a test pipe only records static head. One pipe of a series may enter a firmly compacted pocket through which there is little or no flow, while another pipe in the same series may end in loosely compacted and coarse material, or in a crevice in the rock through which there is considerable and rapid flow. The water column in the first pipe would measure practically total head, while that in the latter would record only the static head. When plotted, the points representing the upward pressures, as measured in these pipes, would lack correlation with each other.

The work of preparing the site for a dam and the laying of the base masonry disturb Nature's placement of the foundation material. Water flowing through the foundation material will seek lanes or paths of least resistance, and these lanes may be neither straight nor at the surfaces of the foundation under the base. Consequently, observations of upward pressures in pipes arranged in a row normal to the axis of the dam may not show consistent relations; nor be such as to indicate a smooth hydraulic grade line. Furthermore, cut-offs will affect the uplift observations and produce results that may appear erratic.

Cut-offs made of sheet-piles may not be water-tight. The piling might be tight for some distance and then have an opening caused by a boulder or other sub-surface obstruction. The path of the water passing through such an opening cannot be predicted.

Deposit of silt will tend to close the voids between the soil particles, and thereby diminish the flow through the foundation. This silting process may divert the flow from some observation pipes and not from others. The silt blanket in a reservoir behind a dam, the crest of which is high above the river bed, will not be disturbed ordinarily by flood discharges, as mentioned by the author* for the Colorado River Dam the crest of which was only 8 ft. high.

Considering the complexity that exists, the writer does not think that the quotients obtained by dividing the distances, as measured along the base of a structure in the direction of river flow, by their corresponding losses of static head (the percolation factors of Bligh), should be proportional in every instance. While the losses of static head may be measured with fair accuracy by the test pipes, the lengths of the paths along which the underground water flows, is not known with any certainty.

The hydrostatic head in the pipes will be affected by the distance the perforations or pipe ends are below the base of a dam. Thus, if one pipe is deep and another shallow, the hydrostatic heads may lack correlation. It is the writer's opinion that under ordinary conditions, the pipe openings should be not less than 18 in. and not more than 6 ft. below the base. This would place the openings of the pipes below the irregularities of the base masonry and still near enough to record the uplift pressures in the layer of foundation material on which the base rests. The chance for consistent and correlating records will be improved by care in placing the pipes.

* *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 686.

The writer's plans for the Sherman Island Dam*, across the Hudson River, near Glens Falls, N. Y., contemplated thirteen rows of three pipes each; that is, one row in alternate bays of the multiple arches, or about 38 ft. apart as measured across the river. Unfortunately, as constructed, only seventeen pipes were placed, with four bays having two pipes, and three bays, three pipes, each. In one bay there was a defective pipe. All of them were on the down-stream side of the 55-ft. interlocking steel sheet-pile cut-off. The foundation material under the dam was sand of varying sizes, containing some boulders. The geologic rock floor, except at the abutments, was not reached by 80-ft borings.

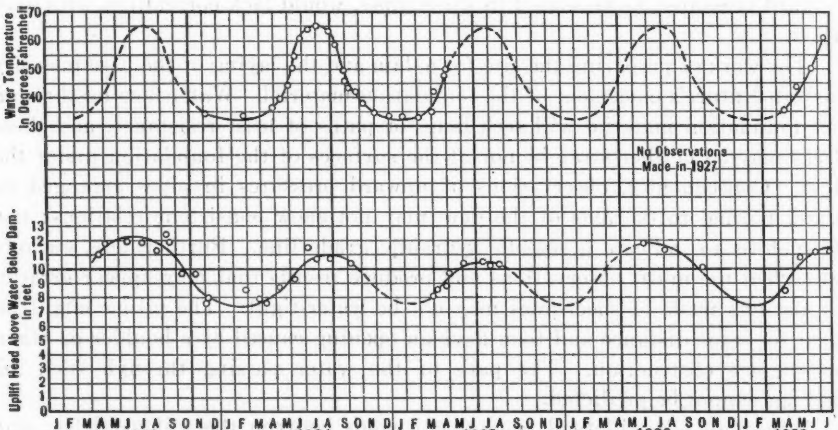


FIG. 31.—SEASONAL VARIATION OF UPLIFT HEAD UNDER MAIN DAM AND WATER TEMPERATURES, SHERMAN ISLAND DAM.

The upward pressures were fairly consistent in the pipes of each row, but were not always correlated when comparing one row with another. The uplift records showed an annual periodic variation which fluctuated with the changes in temperature of the river water. Thus, the average uplift head above water below the dam became greater as the temperature of the water increased, and became less as the temperature lowered. This seasonal variation is shown in Fig. 31. It was noticed that the up-stream cut-off of sheet-piling caused a large diminution in uplift, after which the uplift diminished toward the down-stream edge of the apron along a curved line. The low uplifts for 1925 may be due to low-water temperatures, as no temperature observations were recorded in that year during the summer months.

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), Fig. 18, p. 1287.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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# PAPERS AND DISCUSSIONS

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## THE COMPRESSIBILITY OF SAND-MICA MIXTURES

### Discussion*

By Jacob Feld, Assoc. M. Am. Soc. C. E.

Jacob Feld,† Assoc. M. Am. Soc. C. E. (by letter).‡—This paper is the first discussion of granular materials of non-uniform grains. Earlier discussions of the subject of soil mechanics, with the exception of those by Charles Terzaghi, M. Am. Soc. C. E., have dealt solely with one-shaped grains. In beginning the study of heterogeneous combinations of units which make up that material commonly known as soil, the author has contributed a valuable analytical method of attack to this most important subject, the very basis of all foundation engineering.

In general, this series of experiments deals with mixtures of perfect spherical and perfect flat particles, both of which have a common maximum dimension. There must also be borne in mind that the author has chosen uniform silica particles and uniform mica particles. From the description of characteristics given in the paper,§ it seems that the sand was almost pure silica, one of the hardest and most elastic of minerals, and the mica was a muscovite, which is a potash mica. This type is one of the most common forms of mica and together with biotite (a mica containing iron and magnesia), is characterized by its low water content and high elasticity of the individual flakes.

It is quite possible that a similar combination of, say, spherical limestone particles and flat chlorite particles would give quite different experimental results and conclusions. The limestone which appears in Nature very often is of crystal character and contains considerable water which can be removed either by heat or by pressure. Chlorite is a common mineral found in schists, slates, etc., having the same characteristics as mica as to shape and cleavage. Whereas mica is a common constituent of igneous and metamorphic rocks, chlorite is of secondary origin and is more likely to be encountered in soils. Chlorite gives off considerable water upon the application of heat or pressure. The specific gravity of mica varies between 2.7 and 3.2; that of chlorite varies between 2.65 and 2.96. The only sure method of distinguishing

* Discussion of the paper by Glennon Gilboy, Jun. M. Am. Soc. C. E., continued from August, 1928, *Proceedings*.

† Cons. Engr., New York, N. Y.

‡ Received by the Secretary, June 28, 1928.

§ *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 559.

between these two minerals is the difference in elasticity of the individual flakes.

These data are presented to show that although the results of the paper are valuable in themselves, their application is restricted to the materials used. It would be quite interesting to know whether the author has made any attempt to separate bulky from flat particles by the use of screens with rectangular openings. Such screens could be advantageously used in determining a factor similar in nature to the fineness modulus, but they would take into account two dimensions.

In reference to the average results shown on Fig. 2,* the writer would appreciate any information which may lead to an explanation for the small void ratio under load for the 100% mica sample and at what percentage of mica in the mixture the voids ratio under load decreases. From the diagram it seems that the voids ratio of pure mica under load is smaller than that for any mixture of sand and mica containing more than 5% mica. It is, of course, possible to explain this by the assumption that mica grains in a mixture containing more than 5% of mica are supported on other grains farther apart than in mixtures containing small quantities of mica. Under load, the total deformation (which is a function of the deflection of the individual mica grains acting as beams and struts), would be larger for the greater separation of grains.

It is very interesting to note that the author is in favor of classifying soils by means of pressure-compression and time-compression diagrams in so far as the classification is to be used for structural design. The author argues too rapidly†, however, when he deduces a relative settlement of foundations built on soils containing variable mica proportions. The settlement of a foundation is a function of a considerable number of other variables, and quantitative deductions are liable to serious error.

In his enthusiasm to explain his thesis the author has erred somewhat in stating that particles of colloidal size do not have properties different from materials containing particles of larger size and, also, in the statement that the scale-like grains should be accorded a major consideration in ideas of soil behavior.‡ To explain this contention it is only necessary to remember what a small amount of a material, such as pure colloidal clay, or even a small percentage of water, will do to the physical characteristics of a dry, uniform sand.

The writer agrees with Mr. Gilboy in his final statement that the analysis of soils and their behavior will be greatly simplified when a quantitative formulation of soil types based on numerical constants has been developed.§ It was shown by Helmholz and by Kirchhoff that two constants are sufficient to define an elastic solid completely, namely, the bulk modulus and the shear or rigidity modulus. It can hardly be hoped that at the present state of knowledge of soils, two or even three numerical constants will be sufficient to define any one soil fully.

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 560.

† *Loc. cit.*, p. 564.

‡ *Loc. cit.*, p. 566.

§ *Loc. cit.*, p. 568.

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# PAPERS AND DISCUSSIONS



## SILTING OF THE LAKE AT AUSTIN, TEXAS
### Discussion*

By Messrs. P. A. Welty, Banks McLaurin, R. G. Tyler, and E. C. H. Bantel.

P. A. Welty,† M. Am. Soc. C. E. (by letter).‡—This paper gives testimony as to the seriousness of the problem of silt deposit in the canals and reservoirs of the Southwest. All streams of this region do not carry such excessive quantities of silt, but most of them do, and where this condition does obtain, it seriously affects the operation of canal systems and the life of the reservoir. There are a few reservoirs already constructed in Texas, and others contemplated, which will not be reduced in storage capacity appreciably in several centuries, and there are others built and contemplated that will be greatly reduced in capacity in an average man's life time.

During two periods, 1907 to 1909 and 1916 to 1918, while engaged as an engineer, in the Lower Rio Grande Valley of Texas, on the San Benito Project, the writer had the opportunity to observe the effects of silt on canal operation. The Lower Rio Grande Valley is a delta country that has been built up from the levels of the sea, or the Gulf of Mexico, by the deposit of silt. The river itself flows along a ridge, or backbone, which is (or was until very recently) being constantly raised in elevation along its course, by silt deposited when the river overflows its banks, and also by blow-sand. When these overflows occur, the water leaves the channel and flows across country at right angles, approximately, to the stream. It runs into Mexico on the right side and across the delta plain toward the Gulf of Mexico on the left side in the State of Texas. As these over-flows occur without much velocity, the waters deposit their heavy loads of silt quickly. The heavier grains are dropped first, and the

* Discussion of the paper by T. U. Taylor, M. Am. Soc. C. E., continued from August, 1928, *Proceedings*.

† Chf. Engr., Brown County Water Impvt. Dist. No. 1, Brownwood; San Antonio Suburban Irrigated Farms, Natalia, Tex.

‡ Received by the Secretary, June 18, 1928.

smaller ones next in order, and so on, some of the finer particles being carried until low depressions are reached.

There are many old former river beds of the Rio Grande (locally called "resacas") meandering through the valley, that have been built up in this manner during the past ages, until the time the river reached a point where it broke away from its channel and sought a new flow line through the lower country which, in turn, it began to raise as before. Some of these "resacas" are mentioned in United States history as being the battlegrounds during the Mexican War in 1848.

More than 500 000 acres of land are now being watered in the lower valley of the Rio Grande, in varying areas, by pumping water from the river, making lifts from 12 to 75 ft. to lands above and conducting it to lands embraced in a score or more of projects down the valley as far as Brownsville, Tex., at the lower end of the river. The plans for the San Benito Project, in Cameron County, included low-lift and high-lift pumps, and, in addition, proposed to obtain gravity-flow water for a large percentage of its lands, by taking water directly from the river. The plans were developed and the works constructed by the Chief Engineer, Mr. Sam A. Robertson.

Referring to Figs. 11 and 12, an idea of project locations can be obtained. Fig. 12($a$) is a sketch (not to scale) of the profile of the 2-mile dredged channel conveying the waters of the river to the old "resaca" that was to become the Main Canal of the San Benito System. Fig. 13($b$) shows the typical cross-section of the "resaca", which varied greatly in widths, between banks, from 250 to 1 000 ft. and in depths from 15 to 20 ft. The inside slopes of the "resaca" were from 3 on 1 to 5 on 1. The present river through the valley is more uniform and varies but little. The banks are almost vertical for at least 15 ft. from the top.

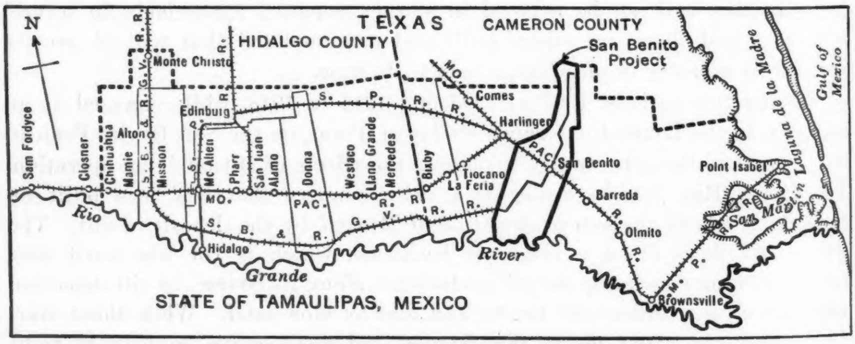



FIG. 11.—GENERAL PLAN, SAN BENITO PROJECT.

Fig. 13($b$) also shows the conditions of the upper few miles after ten years of operation, in the quantity of silt that has been placed in the "resaca", with no thought of maintenance work for its removal.

Fig. 13($c$) shows a sketch profile (of no regular scale) of the "resaca" and the different stages of silt deposit as it progressively built in down stream.

At the point marked $G$ (Fig. 12), heavy service gates were installed to receive and control the waters of the river. These were operated with screw-

lifts and were set in position in massive reinforced concrete walls, carried on both round and Wakefield sheet-piling. The floor of the structure was below the bed of the Rio Grande. The pumping plants were also housed in an adjoining room of this same structure, the suction pipes protruding through the river-front wall.

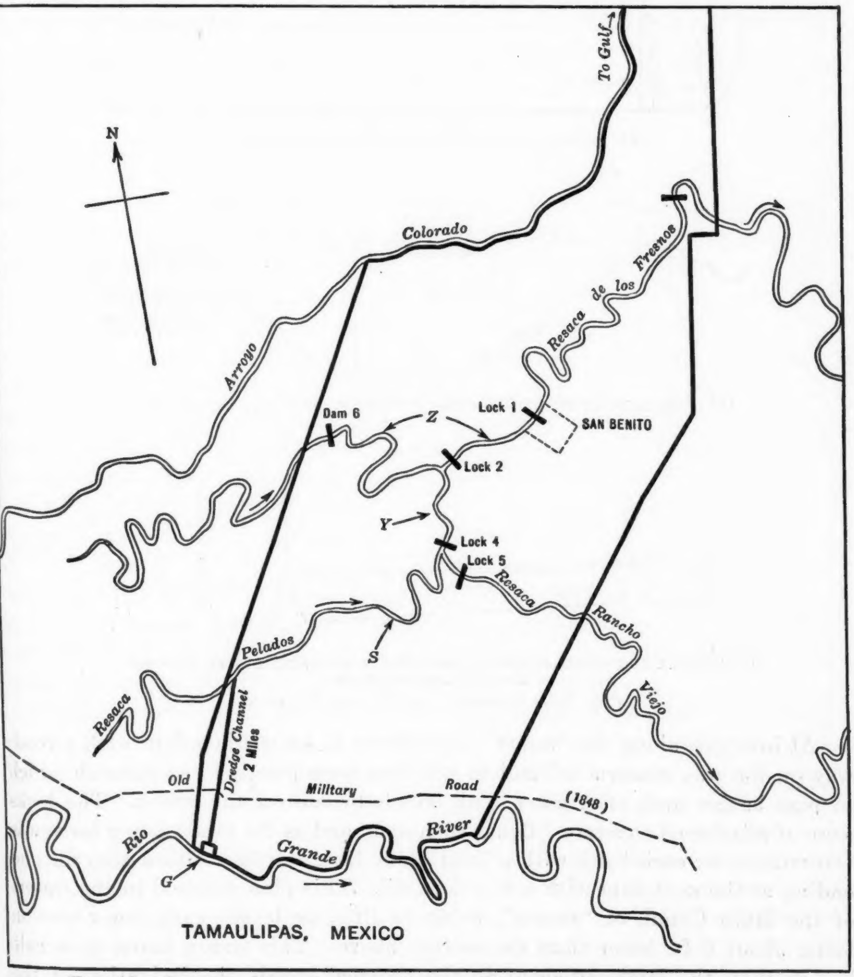
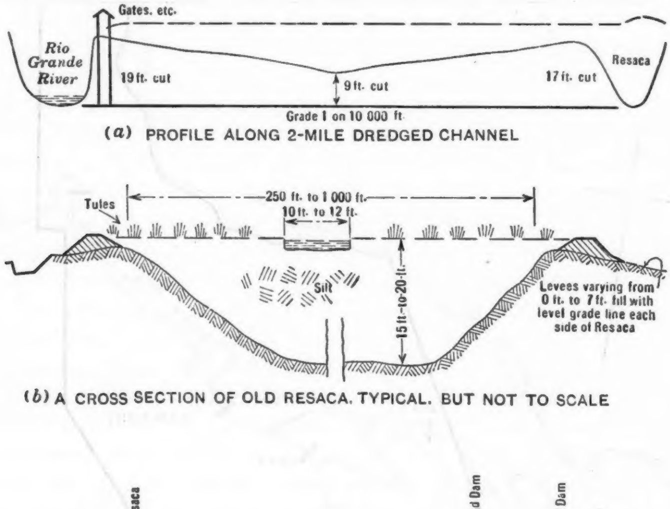



FIG. 12.—OUTLINE MAP, CAMERON COUNTY WATER IMPROVEMENT DISTRICT.

The profile of the dredged channel beginning at *G* (Fig. 13(*a*)), indicates a cut of 19 ft. at the river, as well as one of 17 ft. at the old "resaca", and one of 9 ft. at a point midway between the two. The finished grade of this channel was 1 in 10 000. The channel dredged along this line was 20 ft. wide at the bottom, with side slopes of 1 on 1.

While the land on each side sloped away from the "resaca", the banks of the latter and its bottom also had a uniform slope continually going parallel

to the country, but because of its windings its grade was flatter than a line across country. On the San Benito Project the elevation of the plant shown at $G$ (Fig. 12), was about 48 ft. above sea level, and the Town of San Benito was 32 ft.; the profile then smoothed out to sea level at the Gulf. The land had an average slope of 18 in. to the mile.




(a) PROFILE ALONG 2-MILE DREDGED CHANNEL

(b) A CROSS SECTION OF OLD RESACA, TYPICAL, BUT NOT TO SCALE

(c) PROFILE ALONG RESACA MAIN CANAL SHOWING PROGRESSIVE DEPOSIT OF SILT DOWNSTREAM

FIG. 13.—PROFILE AND TYPICAL SECTIONS.

At intervals along the "resaca", and across it, an earthen dam with a roadway on top was constructed and in the dam were placed locks through which to pass barges and, also, gates with which to control the water. The locks were of reinforced concrete. Below each dam, and as the grade fell, a levee was constructed on each bank with a level grade line, starting with a zero fill and ending at the next dam with a 6 or 7-ft. fill. This plan resulted in the waters of the Main Canal, or "resaca", being in lifts, or levels, each lower section being about 6 ft. lower than the section above. This water, being as a rule higher than the lands adjacent to the "resaca", made the irrigation of the lands easily possible. Laterals were constructed at every ½ mile, or 1 mile, along the "resaca", at right angles, approximately. These served to water lands on either side in the 40-acre units as subdivided. Some lands along the "resaca" were too high to be thus watered, as were the lands adjacent to the river, and high-line canals were located and built along the "resaca" and the river to care for such areas.

In constructing the 2-mile dredged channel it was necessary to take material from borrow-pits to build up the levees near the middle, but at each end

there was excessive waste. This resulted in a channel more than 20 ft. deep. The failure to keep up maintenance work for the removal of this silt soon resulted in this 2-mile channel becoming so full that water could be taken by gravity only when the river was "up" and, later, only when the river was at flood stage. Subsequently, it became necessary to begin the work of raising banks and making what was originally intended to be a low-line canal designed for taking gravity water, into a high-line pumping canal. This was the status when the writer came to the newly organized District on his second engagement there. This old low-line canal was further raised and strengthened so as to form a new high-line canal, and a new low-line canal was excavated through low country, thereby following Nature's plan of leaving the old channel when it had so built up that it could not be followed any more, and then seeking new and lower ground.

The old "resaca" with dimensions as shown in Fig. 13 stored an immense volume of water, and while it was wasteful as regards seepage and evaporation, and was not economical because of the pumping required to fill up the bottom to a height that was usable, yet it was a great land seller and the sight of this great area of water was something that the average prospective buyer could not resist. Consequently, the project grew rapidly until it embraced about 80 000 acres of land, extending down stream along the old "resacas". This steady growth of the project down stream was responsible for the neglect in the upper sections. During the time of the so-called Border trouble, when it seemed as if the Federal Government might intervene in Mexico, there was a period of neglect in the maintenance work caused by re-organizing into the District form of ownership and also bad financing. By 1916, no water was transferred by gravity and pumping was done only at a great additional expense for installing many re-lift pumps at the laterals. Bank-raising was continually in progress, and in order to meet the changing conditions, new pumps of greater capacity were installed at the head-gates.

For the first ten years of operating the system, the quantity of silt deposited in the 2-mile channel and in the first few miles of the upper end of the "resaca", as shown by measurements made by the engineers in February, 1917, reached a total of something less than 5 000 000 cu. yd. The results of the surveys would indicate that the Rio Grande water approaches, in silt carrying capacities, the Colorado River of Texas and also the Colorado River in the Imperial Valley of California. Yet surveys and reports of engineers and project managers of this Texas valley have not thus far considered the Rio Grande in this light. Those interested in the Imperial Valley have had the advantage of gathering more complete data, but little attention has been paid to the seriousness of such matters in Texas.

In Fig. 12, *S*, *Y*, and *Z* are points at which the silt deposit has a decided "drop-off". The heavier particles seem to deposit first (as soon as they strike dead water) and from the river down stream this point had reached to *S* (Fig. 13(*c*)) by 1917. The next gradation of silt, evidently affected by size or some other cause was at Point *Y* and the next section was closer to the town at a point, *Z*. In this section the silt was so soupy and thin as to be almost un-

recognizable in making soundings and, in order to be determined, had to be dipped up.

Below Point *Z* nothing was indicated and the elevations on land lines taken ten years previously checked measurements to the bottom. These sections of silt deposit seem to be almost level and the "drop-offs" at *S* and *Y* are rather abrupt, the change of elevation seeming to occur in ¼-mile intervals along the canal.

The upper reaches of the "resaca" had become silted to the top of the banks. In some cases the banks had to be raised, except along the channel where the water supply flowed, as indicated in Fig. 13(*b*). This channel was about 12 ft. wide and 2 ft. deep, and meandered through the tules and the silt bed. Laborers were constantly engaged in cutting tules, raising banks, and endeavoring to enlarge and keep open this small channel through which the water supply flowed. The velocity was a little better than 2 ft. per sec., which helped to keep moving the silt that was in suspension, and to carry it forward to the points, *S* and *Y*. This silted section of the "resaca" was under water when the pumps were operating and the tules caused the formation of pools of still water in which the silt would drop immediately. Consequently, this silt bed was continually being raised, which, in turn, made it necessary to keep raising the banks.

Some of the projects of the valley used old "esteros", or lakes, that functioned as settling basins. Levees were built around these lakes and the water was discharged into them directly from the pumps, only to be admitted to the canal system after much of the silt had settled out. It was then continually necessary to remove the silt from these basins, and raise the banks.

BANKS MCLAURIN,* ESQ. (by letter).†—The writer has been associated with Dean Taylor on one of the silt surveys of Lake Austin, and has lived on the banks of the Colorado River (Texas) for a life time. He is, therefore, rather familiar with the river, its topography, floods, droughts, and silt-bearing capacity. The Austin Lake stretches its crooked course from Austin into the mountains to the northwest. Those unfamiliar with the lake would conclude that it is a channel reservoir, but this is only partly true. It cannot be classed as a channel reservoir. Fully 50% of its area was formerly on the banks or above the top of the banks. This lake brings into strong contrast the behavior or silting of its channel part and the valley or part that covered former farm lands. Measurements of Lake Austin upset many theories and show conclusively that the valley part of the lake was silted much more rapidly than the channel part. In many places along the 25 miles of lake shore, the silting of the valley part (not in the old river bed) has been more than 100%, and is being increased with every flood. At first, the low-lying banks were only a few feet high, but after each flood, grass, weeds, or willows spring up, and these growths catch trash, drift, etc., retard the velocity of the water, and cause the silt to drop out on the new banks. Slowly but surely the channel width is being narrowed.

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† Received by the Secretary, June 28, 1928.

*Silting of Lake Penick.*—Lake Penick, at Lueders, Tex., presents a case of silting in some respects similar to, and, in other respects, in contrast to, Lake Austin. The reservoir is on the Clear Fork of the Brazos River, in the eastern part of Jones County, Texas, and is the source of water supply for the City of Stamford. The dam is at Lueders, the reservoir extending up the river for about 7 miles. This is more of a channel reservoir. The drainage area covers parts of Jones, Scurry, Fisher, Taylor, Callahan, and Shackelford Counties. The area of the water-shed is approximately 2 250 sq. miles. In this water-shed are the Towns of Abilene, Sweetwater, Buffalo Gap, Roby, Anson, and Merkel. The main stream above the dam is fed by fully a dozen creeks, draining from the south to the north. Fig. 14 shows the location of the dam and reservoir and Fig. 15, the cross-sections on the reservoir.

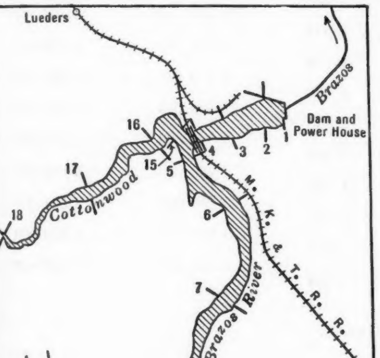


FIG. 14.

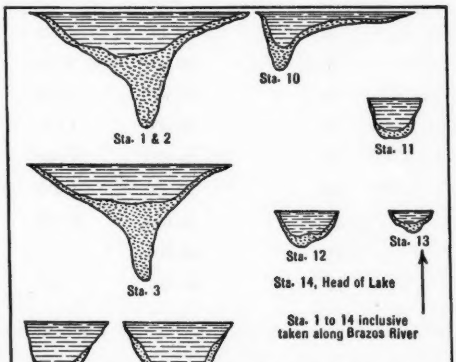


FIG. 15.

The original survey of the reservoir and dam site was made by Mr. Ed Burrow in 1918. Construction on the dam was begun in 1918 and finished in 1920. The cross-sections of 1920 are from Mr. Burrow's field notes, while the

TABLE 5.—VOLUME OF WATER IN LAKE PENICK.

| Section No. | Station distance, in feet. | WATER AREAS, IN SQUARE FEET. | | WATER VOLUMES, IN CUBIC FEET. | |
|---|---|---|---|---|---|
| | | 1927 | 1920 | 1927 | 1920 |
| CLEAR FORK OF BRAZOS RIVER. | | | | | |
| 1 | 00 | 5 500 | 8 000 | 2 750 000 | 4 000 000 |
| 2 | 500 | 5 500 | 8 000 | 8 480 000 | 12 704 000 |
| 3 | 2 100 | 5 100 | 7 880 | 5 376 000 | 7 779 000 |
| 4 | 3 500 | 2 580 | 3 100 | 3 358 000 | 4 510 000 |
| 5 | 4 600 | 3 525 | 5 200 | 10 500 000 | 16 719 000 |
| 6 | 7 100 | 4 875 | 8 175 | 15 320 000 | 24 120 000 |
| 7 | 10 300 | 4 700 | 6 900 | 14 200 000 | 20 800 000 |
| 8 | 14 300 | 2 400 | 3 300 | 7 800 000 | 11 100 000 |
| 9 | 18 300 | 1 500 | 2 250 | 3 685 000 | 4 978 000 |
| 10 | 20 500 | 1 850 | 2 275 | 5 760 000 | 6 849 000 |
| 11 | 24 100 | 1 350 | 1 530 | 3 108 000 | 3 564 000 |
| 12 | 26 500 | 1 240 | 1 440 | 3 680 000 | 4 300 000 |
| 13 | 30 500 | 600 | 710 | 1 350 000 | 1 598 000 |
| 14 | 35 000 | 000 | 000 | | |
| COTTONWOOD CREEK. | | | | | |
| 15 | 0 000 | 2 000 | 4 600 | 2 022 000 | 4 260 000 |
| 16 | 1 200 | 1 370 | 2 500 | 3 738 000 | 5 600 000 |
| 17 | 4 000 | 1 300 | 1 500 | 1 625 000 | 1 875 000 |
| 18 | 6 500 | 000 | 000 | | |
| Total, in cubic feet...... | ........ | ........ | ........ | 92 752 000 | 134 768 000 |
| Total, in acre-feet...... | ........ | ........ | ........ | 2 129 | 3 094 |
| Silt, in acre-feet, 1920-27.. | ....... | ........ | ........ | 3 094 — 2 129 = 965 | |

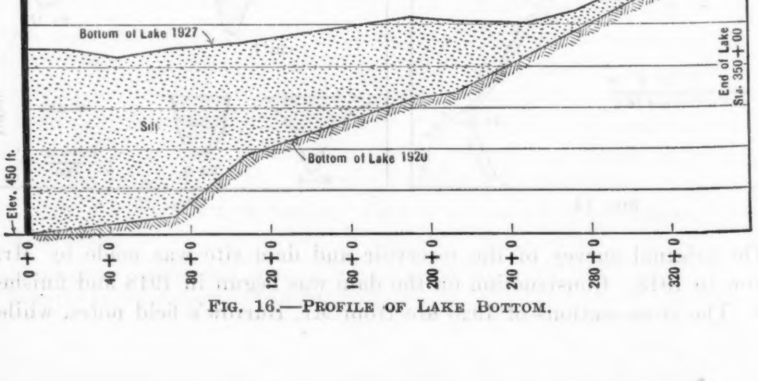


FIG. 16.—PROFILE OF LAKE BOTTOM.

cross-sections of 1927 are from data taken by the writer in August, 1927. In 1920, the water volume of the reservoir was 3 094 acre-ft., while, in 1927, it was 2 129 acre-ft. Thus, the water capacity of 1927 is 68.2% of that in 1920, or, in other words, the lake has silted 31.2% of its original volume in 7 years. These results are shown in Table 5 and in Fig. 15. Fig. 16 shows a longitudinal section taken along the axis of the reservoir from the head of the lake to the dam.

R. G. Tyler,* M. Am. Soc. C. E. (by letter).†—This paper contains information of interest and value to engineers engaged in the construction of reservoirs for water supply or power purposes in the Southwest. The writer has had some familiarity with the local situation and is greatly interested in this most recent chapter in the unusually unfortunate history of the Austin Dam. It will be recalled that, at the time the various engineers were making their investigations prior to the building of the dam, too few data were available as to stream-flow conditions. There are very few data of a reliable nature available at the present time on the quantity of silt carried by streams in various sections of the United States. The Federal Department of Agriculture could render a most commendable service to the Engineering Profession by conducting extended investigations on this problem.

From such data as the writer has at hand, it would appear that one might expect this stream to carry a burden of about 1% of its volume, or about 18 000 acre-ft., of silt per year.‡ It has been estimated that the Brazos River above Waco, Tex., with a drainage area of 30 000 sq. miles, carries 3 200 000 tons of soil per year.‡ These streams are believed by the writer to be somewhat similar, but these estimates give the Colorado nine times the silt burden of the Brazos. The estimate for the Colorado is probably the more accurate as indicated by an analysis of the data given by Dean Taylor.

Table 6 gives the data from *Bulletin No. 1430,* of the U. S. Department of Agriculture, expressed in convenient units. It will be noted that there is an extremely wide variation in the amount of silt carried by streams in different sections of this country. The streams of the northeastern section of the United States are relatively free from silt, due largely to the nature of the tributary area.

The character of the drainage area is an important factor with reference to its silt production. With similar soil conditions, open, cultivated tracts erode more than wooded or grazing areas. In arid areas, also, where high temperatures and low rainfall discourage the growth of vegetation, the turf provides inadequate protection against erosion. These conditions prevail over a large part of the Colorado River drainage area above Austin, and are largely responsible for its high silt burden. This is a condition which has a greater effect as time goes on, due to the increased acreage coming under cultivation. Such erosion is objectionable both from the standpoint of impoverishing the denuded areas and of silting streams and reservoirs. The conservation of

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† Received by the Secretary, July 13, 1928.

‡ *Bulletin No. 1430,* U. S. Department of Agriculture, p. 24.

forests and the reforestation of denuded areas should have a beneficial effect in decreasing siltation difficulties.

TABLE 6.—SILT CONTENT OF VARIOUS RIVERS.

| Stream. | Tons per year. | Drainage area, in square miles. | Tons per square mile per year. |
|---|---|---|---|
| Brazos River, Texas | 3 200 000 | 30 000 | 107 |
| Missouri River, above Ruegg, St. Louis County, Mo. | 176 000 000 | 528 700 | 333 |
| Arkansas River, above Little Rock, Ark. | 40 000 000 | 148 000 | 270 |
| Colorado River, above Austin, Tex. | 32 500 000* | 34 200 | 950 |
| Tennessee River | 11 000 000 | 35 000 | 314 |
| Yadkin River, above Salisbury, N. C. | 850 lb. soil per year per acre. | ....... | 272 |
| Mississippi River, above Minneapolis, Minn. | 117 000 | 19 585 | 6 |
| Susquehanna River, at Danville, Pa. | 240 150 | 9 530 | 25 |
| Hudson River | 240 000 | 13 366 | 18 |

* Estimated as 18 000 acre-ft.; 50% voids assumed.

Assuming that a silt burden of 18 000 acre-ft. per year is a fair value for the Colorado River, it is evident from the measurements given by the author that the quantity deposited is much less than this. For example, the average deposition per year between 1893 and 1900 was about 3 366 acre-ft., while the first year of this period may have yielded about 7 800 acre-ft., if a distribution similar to Fig. 5* is assumed. During the period, 1913 to 1922, the average deposition was 2 962 acre-ft. per year; while, during 1922 to 1924, it was 1 230 acre-ft. annually, with only 712 acre-ft. per year between 1924 and 1926. Again (see Fig. 5), the maximum silt deposit occurring in 1913 was about 6 400 acre-ft. Thus, an ever greater proportion of silt is carried over the dam each year, due to the decrease in the period of detention in the lake and to the corresponding increase in the velocity of water flowing through. The shape of the reservoir with its relatively small transverse section is such as to make the silting less than would occur in the more usual type, with greater lateral dimensions. This shape affects the velocity, which effect must be considered in predicting the amount of siltation.

The topography at the location of the dam has been described in various reports and papers, but there is one feature that is worthy of further comment. The valley down stream from the dam gradually widens and considerable quantities of coarse sand and gravel, comparatively free from silt, have been deposited in its upper reaches, from which material has long been taken for concrete aggregate for local construction work. This area extends to a point opposite or below the city proper and, as the valley continues to widen, the sand becomes finer and contains greater quantities of silt, so that a few miles farther down stream there is a broad valley where the soil consists of a sandy loam used for the growing of garden produce. This indicates the character of sediment carried by the stream and deposited at the point where the channel emerges from the canyon section into the valley section and the gradient flattens out from 4.2 to 1.7 ft. per mile.† An examination of this valley would

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 577.
† *Water Supply Paper No. 44*, U. S. Geological Survey.

give the engineer a very good idea of the type of material which the stream might be expected to bring down at flood stage. The paper indicates a somewhat similar arrangement of materials in the lake, with its sand deposits for a distance of about 3 miles above Station 17.75 and grading off into silt below that point.

In constructing reservoirs in semi-arid regions, the engineer is confronted with the problem of providing a volume sufficient to allow for silting and still give a reasonable life that, at the same time, provides a large surface area from which excessive evaporation losses occur. The writer has found cases where, with the low rainfall and percentages of run-off and the high evaporation from water surfaces encountered in the Southwest, it has been difficult to find reservoir sites which would supply a reasonable demand. The difficulty is more acute where the valleys are shallow and the reservoir volume is distributed horizontally rather than vertically. In such reservoirs there is considerable shoal water which adds very little to the available storage, but from which a depth of water of 50 to 70 in. is lost in evaporation annually. Large volume, therefore, is not necessarily a solution for the silt problem, as it introduces another problem quite as troublesome.

In some instances, especially abroad, siltation has been controlled by the construction of an upper reservoir with a by-pass around the lower or main reservoir. The water carrying the higher turbidity is by-passed and only the partly settled water is carried into the lower reservoir. A by-pass would have been out of question in the present instance, however, because of the topography.

E. C. H. Bantel,* M. Am. Soc. C. E. (by letter).†—In this paper Dean Taylor calls attention to the important problem of providing adequate water supplies for the growing cities and towns in the State of Texas, and for other States having agricultural and manufacturing interests.

The State Board of Water Engineers, which has supervision over the flowing waters of the State, has declared that all the ordinary flow of all the rivers and streams of the State has been allotted. Permits have been issued to individuals, corporations, cities, and towns, authorizing them to take from the rivers and streams quantities of water which, in the aggregate, equal their total ordinary flow. Additional permits cannot be granted. The supply is exhausted. Only the storm waters remain unallotted. These are regarded as the property of the State to be disposed of as may be decided by law.

It is also true that at times the ordinary flow of the streams is insufficient to supply present needs. The irrigation companies are constantly clamoring for more water, and many cities and towns have suffered long and severe shortages. Abilene, Dallas, Dalhart, and Elgin may be mentioned in this connection. The population of the State and its agricultural and manufacturing interests are growing steadily and the demand for water is growing proportionately. If an adequate supply is not provided, growth in population and industries will soon cease.

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† Received by the Secretary, July 21, 1928.

TABLE 7.—QUANTITY OF SILT AND RATE OF SILTING IN RESERVOIRS IN TEXAS AND NEW MEXICO.

| State. | Authority. | Reservoir. | Drainage area, in square miles. | Original capacity, in acre-feet. | Period. From. | Period. To. | Number of years in period. | Silt accumulated, in acre-feet in period. | Silt accumulated, in acre-feet per year. | Silt accumulated per square mile of water-shed. |
|---|---|---|---|---|---|---|---|---|---|---|
| Texas | T. U. Taylor | Old Lake Austin | 38 000 | 49 300 | 1893.3 | 1900.1 | 6.8 | 23 560 | 34 600 | 0.091 |
| | T. U. Taylor | New Lake Austin | 38 000 | 32 029 | 1918 | 1922 | 9 | 26 667 | 2 963 | 0.078 |
| | | | | | 1922 | 1924 | 2 | 2 461 | 1 230 | 0.032 |
| | | | | | 1924 | 1926 | 2 | 1 424 | 712 | 0.019 |
| | Charles Schulz | McKinney | 1.56 | 12 | ...... | ...... | 10 | 12 | 1.2 | 0.770 |
| | Banks McLaurin | Lake Penick, Stamford | 2 250 | 3 094 | 1920 | 1927 | 7 | 965 | 138 | 0.061 |
| | J. B. Hawley | Lake Worth | 1 800 | ..... | ...... | ...... | ...... | ...... | ..... | 0.500 |
| New Mexico | L. M. Lawson | Elephant Butte | 30 000 | 2 638 000 | 1916.8 | 1925.6 | 8.8 | 177 740 | 20 200 | 0.670 |
| | H. F. Robinson | Zuni | 650 | 10 230 | 1906.5 | 1910.5 | 4.00 | 1 804 | 451 | 0.690 |
| | | | | | 1910.5 | 1912 | 1.50 | 1 340 | 894 | 1.38 |
| | | | | | 1912 | 1914 | 2.00 | 1 056 | 528 | 0.810 |
| | | | | | 1914 | 1918 | 4.00 | 1 307 | 327 | 50. |
| | | | | | 1918 | 1919 | 1.00 | 678 | 678 | 1.04 |
| | | | | | 1919 | 1920 | 1.00 | 1 248 | 1 248 | 1.92 |
| | U. S. Reclamation Service | McMillan | 22 000 | 29 000 | ..... | ...... | 10.00 | 12 200 | 1 220 | 0.055 |

In a large section of the State artesian water is available and will be developed as needed, but in a much larger part there is none. Conservation and use of the now unappropriated flood waters is the only recourse left to this region. Conservation of flood waters implies the construction of reservoirs, and experience shows that these become filled with silt and are useless in comparatively short times. Table 7 shows the quantity of silt and the rate of silting in some reservoirs in the Texas and New Mexico sections.

The silt accumulation, in acre-feet per square mile of drainage area per year, is given in each case. The quantity for Lake Austin varies from 0.097 to 0.019 acre-ft. The average quantity for the 20 years is about 0.07 acre-ft. Compared with the figures for the other reservoirs listed in Table 7, this quantity is small, but the new Lake Austin has been practically filled with silt in fifteen years. During the years when the capacity of the reservoir was great, the quantity of silt stored was also great. As the capacity of the reservoir became reduced by silt accumulation, affording less opportunity for sedimentation, the quantity deposited per square mile of drainage area steadily decreased. This seems natural and logical, but there is no such decrease in the silting of the Zuni Reservoir. Here, the rate of accumulation was greatest when 70% of the original capacity of the reservoir was filled with silt. It is interesting, also, to note that the rates of accumulation in the Elephant Butte and Zuni Reservoirs are much greater than in Lake Austin. The rate for Elephant Butte Reservoir is seven times, and that for Zuni Reservoir varies from seven to twenty times, the maximum rate for Lake Austin. McKinney Reservoir has a high rate of accumulation, 0.77 acre-ft. per sq. mile of drainage area per year.

The method of preventing silting that has shown promise of success is to build dams with under-sluices through which the silt-laden flood waters may be passed. This method is said to be successful in India and Egypt. It may be that it can be adapted to Texas conditions.

# AMERICAN SOCIETY OF CIVIL ENGINEERS

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## PAPERS AND DISCUSSIONS

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# CONTINUOUS BEAMS OVER THREE SPANS

## Discussion*

By W. F. Way, M. Am. Soc. C. E.

W. F. Way,† M. Am. Soc. C. E. (by letter).‡—Lest there might be created in the minds of some designers, the opinion that after making his particular design according to the moment factors given by the author, that the results represented the actual stresses likely to occur in the continuous member in question and that there was nothing left to do, it might be well to consider the fundamental principles on which these factors are founded and to compare them with the actual existing conditions.

This criticism would not be made if the writer thought that it would in any way tend to lessen the amount of theoretical work done on continuous members in reinforced concrete. Good design in continuous beams cannot be made without first exhausting all theoretical solutions applicable to the case in question. To the extent that Mr. Oesterblom has aided these solutions, he is to be given credit. The following comments apply to reinforced concrete construction because at present it is only in this material that there is any need for the application of this theory.

In determining his moment factors Mr. Oesterblom has assumed:

1.—That the concrete beam was resting freely and unrestrained on knife-edge supports.
2.—That the beams are homogeneous, with constant moment of inertia and modulus of elasticity.
3.—That the stresses in the beams are unaffected by shrinkage.
4.—That the temperature stresses are negligible.

Consider these assumptions in order to see how nearly they parallel actual concrete construction.

Instead of the beam resting freely on a knife-edge support, more often it is resting on a building column and frequently on haunches. In addition,

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* Discussion on the paper by I. Oesterblom, M. Am. Soc. C. E., continued from August, 1928, *Proceedings*.

† Constr. Engr. for Henry & McFee, Seattle, Wash.

‡ Received by the Secretary, June 30, 1928.

there is often a column above the beam continuous with the support. In this case the author's moment factors are certainly altered, the amount depending on story height, spans, and moments of inertia of columns and beams. In a few instances these factors may be small enough to be neglected, but not in many; more often they will alter the factors considerably. Building codes and most textbooks alike neglect this case, the only one suggesting a method for the amount of negative moment in a continuous beam over a wide support, that has come to the writer's attention, is given in "Beton Kalender."* In important and repeat work this method should be only one of several used in arriving at a fair average for the proper figure for design.

Referring to the second assumption, it is true that a reinforced continuous concrete beam is not composed of a homogeneous material, nor has it a constant moment of inertia or modulus of elasticity. In most instances, the difference between this assumption and actual conditions is probably so small that it may be neglected. However, the facts should be realized as it is not improbable that they may materially affect the moment factors in special cases.

It is well known that concrete shrinks considerably during the early period of hardening. This shrinkage induces considerable compression in the reinforcing steel—the amount depending on the steel percentage, cement used, water content of concrete, etc. In a precast concrete project† the writer set gauge points in the main reinforcement for forty beams before they were cast, and observed the compression induced into the steel due to shrinkage of the concrete while hardening. In these beams, tested in this manner, the compression amounted to an average of 12 000 lb. per sq. in. at the end of 60 days. Even after placing these beams in the structure and pouring the slab, thus placing full dead load on them, the bars were still in compression.

There is even another aspect to this property of reinforced concrete which probably affects the stresses in continuous beams more than in the case mentioned. For example, assume a tall building of the apartment type, in which the continuous beams are supported by two wall and two interior columns. The usual construction is a large column forming part of the exterior wall and very lightly reinforced, while to save space the interior column is made as small as possible with a high steel percentage. This difference in steel percentage will result in a difference in the shrinkage coefficient. Since this coefficient depends on the brand of cement used, the water content in the poured concrete, etc., it is difficult to assign a general value for this difference, but according to published results it might be taken as 0.0003. This means that for every 10-ft. story the wall column will shorten 0.036 in. more than the interior columns. This is a small difference but in a 7-story building it amounts to a total of ¼ in. Even this is not realized in buildings, but it must be acknowledged that some force is required to reduce this difference and that this same force is altering the stresses in the continuous beams tying the columns together, which added to the stresses induced into the continuous beams, due to difference in column shrinkage, would be of sufficient magnitude to alter the theoretical moment factors considerably.

* 1923 Edition, p. 305.

† *Engineering News-Record,* March 13, 1924.

As for temperature stresses in reinforced concrete building construction, the effect is similar to the case of shrinkage stresses. Take the apartment house example again and assume the exterior walls to be of concrete, stuccoed. During the winter season the difference between the temperature of the interior and exterior columns may easily be as much as 60° Fahr., which would mean that for every 10-ft story there would be a difference of 0.043 in. in the expansion between the interior and exterior columns. During the summer seasons this condition might easily be reversed. The changing elevations of the supports of the continuous beams will most certainly alter the moment factor, and the temperature stresses added to the shrinkage stresses will most likely be of measurable magnitude.

In the closing paragraphs of the paper* severe criticism is made of the report of the Joint Committee on Specifications for Concrete and Reinforced Concrete, principally because no definite regulation was made for "continuous beams with unequal spans". A similar criticism might well be made of the author because he has offered no solution whatever for continuous beams with fixed ends. In general construction it is seldom that continuous beams with free ends occur. They are usually at least partly fixed if not entirely, and it is quite as important to have a solution for one case as for the other.

Since it is impossible to make an accurate mathematical design for a reinforced concrete beam, empirical factors must be used for building codes. Committees writing these codes have realized this and, as a member of the Seattle Building Code Committee, the writer felt no chagrin at Mr. Oesterblom's severe criticism.

Reinforced concrete buildings have been built and are still being constructed that have and will give satisfaction. It is not the writer's intention in the least to discredit properly designed and constructed reinforced concrete structures. It is the intention of this discussion to show that the assumptions on which Mr. Oesterblom based his calculations for his moment factors are far from being realized in actual construction in reinforced concrete. It is well that these limitations should be clearly understood by the designer and while it may be impossible to solve their numerical influence exactly, still this influence can at least be sensed by the designer and some provision made in the finished structure to carry these stresses.

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* *Proceedings,* Am. Soc. C. E., March, 1928, Papers and Discussions, p. 724.

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# PAPERS AND DISCUSSIONS



## HYDROSTATIC UPLIFT IN PERVIOUS SOILS

### Discussion*

By Messrs. Martin J. McPike and Jacob Feld.

Martin J. McPike,† Assoc. M. Am. Soc. C. E.—Previous to the construction of Dry Dock No. 4 at the U. S. Navy Yard, Brooklyn, N. Y. (1908-12), some difference of opinion was found regarding the amount of hydrostatic uplift which would have to be considered in the analysis of the dry-dock design. As the hydrostatic uplift, if considered, would be a factor of prime importance and an item of some monetary significance, the question of its amount was given quite extensive thought.

Frederick R. Harris, M. Am. Soc. C. E., was at the time in charge of this project, and his experience on structures in water-bearing soil had led him to the opinion that the full hydrostatic head would have to be considered.

In view of the varied opinions expressed at that time and in order to have something tangible on which to base the design of future structures of this type, Admiral Harris had designed and placed at several positions in the floor of this dock, pressure indicators, or, as they were called, dynamometers, of the type shown in Fig. 11.

These pressure indicators were of bronze and consisted of an extra-heavy pipe, 4 in. in diameter and approximately 5 ft. 8 in. long, connected to castings at the upper and lower ends. The lower casting was bell-shaped and had a diaphragm of lead, 12 in. in diameter, weighing 5 lb. per sq. ft. attached to its bottom, by a bolted ring. Bearing against the upper side of this diaphragm and capable of being backed away from the face thereof by a capstan screw extending to, and operated at, the extreme upper end of the dynamometer, was a platen plate with a vertical and horizontal movement that was limited by three lugs into which set-screws, threaded into the bottom casting proper, projected.

* Discussion on the paper by H. deB. Parsons, M. Am. Soc. C. E., continued from August, 1928, *Proceedings*.

† Assoc. Civ. Engr., U. S. Navy Yard, Brooklyn, N. Y.

The top casting contained a stuffing-box or gland, to prevent leakage of the pressure-transferring fluid with which the dynamometer was filled, and formed a connection for the pressure-reading gauge and a pipe for filling the dynamometer.

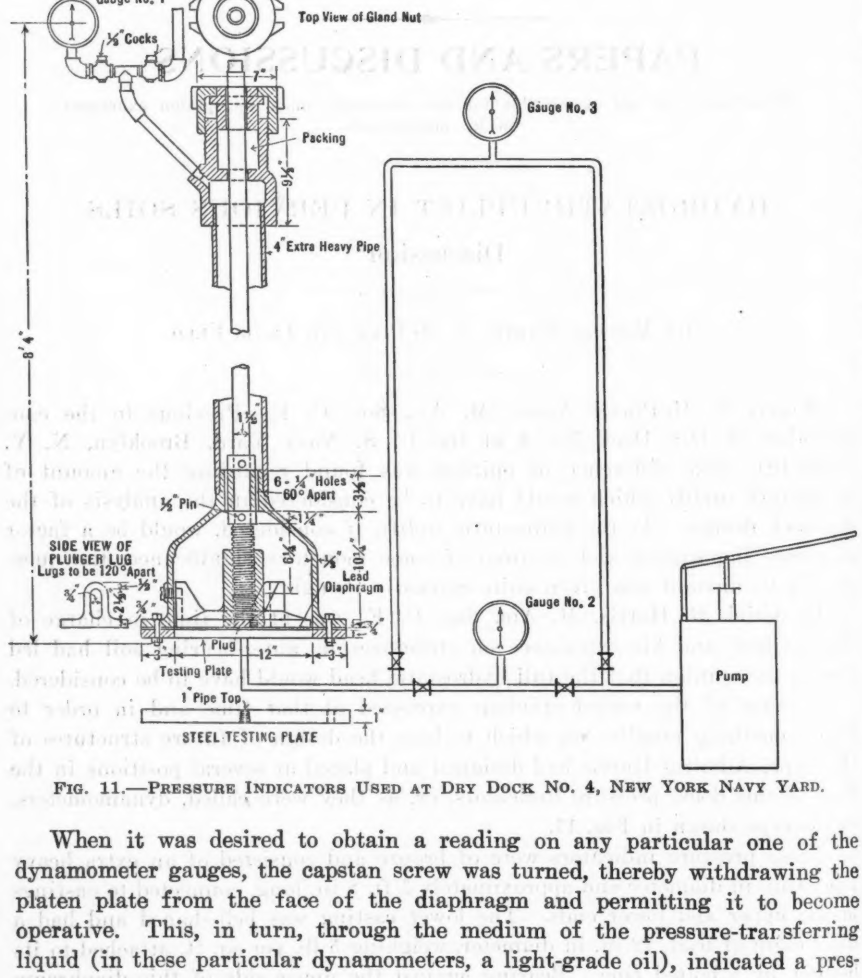


FIG. 11.—PRESSURE INDICATORS USED AT DRY DOCK NO. 4, NEW YORK NAVY YARD.

When it was desired to obtain a reading on any particular one of the dynamometer gauges, the capstan screw was turned, thereby withdrawing the platen plate from the face of the diaphragm and permitting it to become operative. This, in turn, through the medium of the pressure-transferring liquid (in these particular dynamometers, a light-grade oil), indicated a pressure on the gauge.

As regards location in the completed structure, the dynamometers were cast in the concrete floor of the dry dock during its construction, the lead diaphragm being placed in line with the bottom line of the dock floor, the main body of the dynamometer projecting up through the floor, about 8 ft., with the top ending in a pocket just below the upper level of the finished concrete.

In order that the readings on the gauges at the top level of the floor would be truly representative of the pressures encountered at the diaphragm level,

approximately 8 ft. below, it was necessary to adjust the gauges used in the final installation to take care of this difference in head; hence, a series of tests was run on the dynamometers to determine as accurately as possible the "instrument factor" of each, and what loss, if any, would have to be considered in the transmission of the pressure from this lower level to the upper level.

The tests consisted in applying a variable pressure on the bottom of the diaphragm by placing a plate (see Fig. 11) below it, into which was tapped a 1-in. pipe connected to a hand-operated booster pump at the other end. The diagram gives a general idea of the arrangement of pump, gauges, etc., for the tests.

A standard gauge was used in conjunction with a commercial gauge, one being located on the pump (Gauges Nos. 2 and 3), the other being located on the dynamometer (Gauge No. 1), and at every 5-lb. increment from 0 lb. to 30 lb. and every 5-lb. decrement from 30 lb. to 0 lb. indicated on the booster-pump gauge, an observation was taken on the dynamometer gauge.

Each test consisted of ten individual runs, the readings on the dynamometer gauge were recorded, and the averages of each test were used for the final graduation of the dynamometer gauges.

Both lead (5 lb. per sq. ft.) and copper (No. 27 B.w. g.) diaphragms were used in the tests, but lead diaphragms were selected for the completed structure, owing to the most consistent results being obtained therefrom in the tests.

TABLE 9.—TEST NO. 4 WITH DYNAMOMETER NO. 1.*

| Run No. | Pressure Increasing, in Pounds. | | | | | | Pressure Decreasing, in Pounds. | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 5 | 10 | 15 | 20 | 25 | 30 | 25 | 20 | 15 | 10 | 5 |
| 1 | 4.0 | 8.0 | 13.0 | 18.5 | 24.0 | 29.0 | 24.5 | 19.0 | 14.0 | 9.5 | 5.0 |
| 2 | 5.0 | 9.5 | 14.0 | 19.0 | 24.0 | 28.5 | 23.5 | 19.0 | 14.0 | 10.0 | 5.0 |
| 3 | 5.0 | 9.5 | 14.0 | 18.5 | 24.0 | 28.5 | 24.0 | 19.0 | 14.0 | 10.0 | 4.5 |
| 4 | 4.5 | 9.0 | 14.0 | 18.5 | 24.0 | 29.0 | 24.0 | 19.0 | 14.0 | 9.5 | 4.5 |
| 5 | 4.5 | 9.5 | 14.0 | 18.5 | 24.0 | 29.0 | 24.0 | 19.0 | 14.0 | 10.0 | 4.5 |
| 6 | 4.5 | 9.0 | 14.0 | 19.0 | 24.0 | 29.0 | 24.0 | 19.0 | 14.0 | 9.5 | 4.5 |
| 7 | 4.5 | 9.0 | 14.0 | 18.5 | 24.0 | 29.0 | 24.0 | 19.0 | 13.5 | 10.0 | 4.5 |
| 8 | 4.5 | 9.5 | 14.0 | 18.5 | 24.0 | 29.0 | 24.0 | 19.0 | 14.0 | 10.0 | 4.5 |
| 9 | 4.5 | 9.0 | 14.0 | 18.5 | 24.0 | 29.0 | 24.0 | 19.0 | 14.0 | 9.5 | 4.5 |
| 10 | 4.5 | 9.5 | 14.0 | 19.0 | 24.0 | 29.0 | 21.0 | 19.0 | 14.0 | 9.5 | 4.5 |
| Average. | 4.55 | 9.15 | 13.90 | 18.65 | 24.0 | 28.90 | 24.0 | 19.0 | 13.95 | 9.75 | 4.60 |

* Standard gauge at top of loop; commercial gauge on dynamometer; lead diaphragm.

A typical test run is indicated in Table 9. In order that the final results should be consistent under all conditions likely to be encountered in the field, a great deal of care was exercised in running the tests, the position of the gauges varying for the different tests as indicated in Fig. 11 and Table 10. The averages of all tests run, as well as the variation in readings in the dynamometer gauge under the different conditions of gauge location and diaphragm material, are also given in Table 10.

TABLE 10.—AVERAGES OF READINGS ON DYNAMOMETER GAUGE WITH VARYING PRESSURES OF PUMP GAUGE.

| Test No. | Dynamometer No. | Diaphragm material. | Location of standard gauge. | Pressure Increasing, in Pounds. | | | | | | Pressure Decreasing, in Pounds. | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 5 | 10 | 15 | 20 | 25 | 30 | 25 | 20 | 15 | 10 | 5 |
| 1 | 3 | Lead, 5 lb. | Bottom of loop | 3.25 | 7.80 | 12.65 | 17.00 | 22.05 | 27.00 | 22.87 | 17.94 | 13.69 | 9.19 | 4.00 |
| 2 | 2 | " " | " " " | 1.15 | 6.75 | 11.30 | 15.40 | 20.50 | 25.35 | 21.30 | 16.05 | 12.05 | 7.25 | 1.80 |
| 3 | 2 | " " | Top of loop | 4.30 | 8.80 | 13.55 | 18.10 | 23.10 | 28.10 | 23.20 | 18.25 | 13.60 | 9.65 | 4.70 |
| 4 | 1 | " " | " " " | 4.55 | 9.15 | 18.90 | 18.65 | 24.00 | 28.90 | 24.00 | 19.00 | 13.95 | 9.75 | 4.60 |
| 5 | 1 | " " | On dynamometer | 5.00 | 10.00 | 15.00 | 20.00 | 25.00 | 30.00 | 25.00 | 20.00 | 15.00 | 9.95 | 5.00 |
| 6 | 4 | " " | Top of loop | 4.15 | 9.35 | 14.10 | 18.20 | 23.75 | 28.20 | 23.95 | 18.40 | 13.98 | 9.50 | 4.50 |
| 7 | 4 | " " | On dynamometer | 4.70 | 9.45 | 14.45 | 19.35 | 24.15 | 29.05 | 24.20 | 19.15 | 14.45 | 9.15 | 4.60 |
| 8 | 4 | No. 27 B. w. g. copper | Top of loop | 4.40 | 9.53 | 14.45 | 19.40 | 24.30 | 29.15 | 24.38 | 19.68 | 14.75 | 9.98 | 5.00 |
| 9 | 4 | " " " | On dynamometer | 4.78 | 9.78 | 15.00 | 20.00 | 25.00 | 30.00 | 25.10 | 20.05 | 15.20 | 9.90 | 4.98 |
| 10 | 1 | " " " | Top of loop | 4.93 | 9.98 | 14.80 | 19.78 | 24.65 | 29.53 | 24.48 | 19.58 | 14.55 | 10.00 | 4.98 |
| 11 | 1 | " " " | On dynamometer | 4.73 | 9.83 | 15.05 | 20.08 | 25.08 | 30.00 | 25.18 | 20.00 | 15.23 | 9.90 | 4.93 |

After the completion of the dry dock in 1912, observations were taken on these dynamometer gauges four times daily for a period of several months, and their readings, plotted in conjunction with tidal observations, indicated practically 100% hydrostatic uplift fluctuating with tidal changes.

After the dynamometers had served the purpose for which they were installed, the gauges were removed and the pockets filled with concrete. In 1918, two of the dynamometers were uncovered, and observations taken thereon were consistent with the results previously obtained. On June 15, 1928, the speaker again had the opportunity of testing the effectiveness of two of these dynamometers, and they indicated a pressure of exactly the same intensity as they did in 1918.

JACOB FELD,* ASSOC. M. AM. SOC. C. E. (by letter).†—This paper should settle, once and for all, the disputed question of the transmission of hydrostatic pressure through pervious soils. The contentions of some engineers that the area of a wall covered by grains of soil was not affected by the pressure of the water surrounding such grains had no theoretical basis and was never previously tested. The sole basis of the contention was the fact that dams and other structures built on such an assumption had not failed. These experiments prove definitely that the hydrostatic pressure against a surface cannot be decreased by placing a granular material against that surface.

The method of drawing conclusions from the first set of tests‡ is open to criticism. Table 1,‡ giving averages of friction tests, in pounds per square inch, on the side of the can in contact with sand, is purely an arbitrary set of figures which cannot be applied to any other case. A factor which will influence these values is the height of water overlying the top of the sand. The friction of the can embedded in sand, occurring as it does at the point of motion, is a function of the pressure of the sand against the can. The amount of this pressure depends on the depth and the nature of the sand, and the latter depends on the water overlying it. As a result the use of Fig. 3§ to determine the percentage of base area on which hydrostatic uplift acts, gives results which are not quite consistent. In analyzing the data in Table 2‖ it is interesting to note that the average percentage of effective area on which hydrostatic uplift acts is 95.7% for the metal can 15.09 in. in diameter, 88.5% for the 9.22-in. can, and 86.7% for the 5.53-in. can. This is for the condition of wet sand packed in the container. The other set of experiments in the first part, enumerated in Table 3,¶ shows a number of low values for the 9.22-in. can when the sand is packed dry in the container with, later, an addition of water. This can had a plaster-coated bottom and it is highly probable that there was a considerable quantity of air trapped on the surface of the bottom. However, the data from the other two sets of tests are so conclusive that there is no doubt of the validity of the results.

* Cons. Engr., New York, N. Y.

† Received by the Secretary, July 5, 1928.

‡ *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 945.

§ *Loc. cit.*, p. 946.

‖ *Loc. cit.*, p. 947.

¶ *Loc. cit.*, p. 948.

The author states* that he found no appreciable difference in friction due to the materials of which the cans were made, namely, glass, tin, or galvanized iron. The writer believes that in all work on sand and similar soils the same conclusions can be drawn, because the friction between materials like glass, tin, galvanized iron, or wood and sand, is practically the same as the friction of sand against sand. As a result, the true frictional value measured is practically that of sand against sand since a thin layer of the filling material adheres to the test surface and true motion is not between the filling material and test surface, but is a shearing deformation in the filling material itself. Similar results and conclusions were obtained by the writer in his experimental work on the determination of the lateral pressure of soil against walls made of wood, glass, and sheet metal.† Similar results were obtained by Müller-Breslau in 1906 in his experimental work in earth pressures. Certainly the friction between sand and the ordinary concrete or stone used for dams is greater than the friction between sand and such test materials, and the conclusion therefore applies to all actual conditions.

In reference to the tests with clay, the lack of results is explained in the paper by the fact that the clay became liquid in nature. Under such conditions the pressure exerted is given by the formulas in hydrostatics and the same conclusions given for sand then apply to clay. That liquid materials, such as a concrete mix, obey the laws of hydrostatics has been proved by the experimental work of Messrs. F. R. Schunk‡ and E. B. Smith.§

Referring to the experiments of the first set, the amount of friction measured is an indication of the total lateral force acting against the submerged can. The ratio between the vertical force (resistance measured in the tests) to the horizontal pressure of the saturated sand against the can must equal the coefficient of friction between saturated sand and the material of which the can was made. This coefficient can be experimentally determined and is quite constant for all the usual materials, even for such unlike materials as wood, tin, and glass. The results are then directly applicable to the design of walls, quays, or docks acted upon by saturated soils and also to dams against which silt has accumulated to considerable heights. The writer would like to know whether any designer of dams has investigated the change in pressures sustained by a dam because of silt accumulation in the reservoir. It is hoped that, in his closing discussion, the author will explain in greater detail the experimental data obtained, especially as to the conditions of the sand and water levels in the first set of tests.

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 945.

† *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), pp. 1472, 1476, and 1477.

‡ *Professional Memoirs*, U. S. Corps of Engrs., Vol. 1, No. 3, 1909, pp. 247–260.

§ *Proceedings*, Am. Concrete Inst., 1920.

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PAPERS AND DISCUSSIONS
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# ONE HUNDRED FIFTY YEARS ADVANCE IN STRUCTURAL ANALYSIS

## Discussion*

By Messrs. S. Timoshenko, Alfred D. Flinn, and Jacob Feld.

S. Timoshenko,† Esq. (by letter).‡—It is a consequence of the ancient structural project at Babel (as the story goes), that linguistic barriers have hampered the progress of science. These barriers have become of slight concern to the English speaking peoples with reference to many of the languages, especially French and German, but, unfortunately, the Russian barrier is not so easily surmounted. The writer takes this opportunity to help remedy this, and to elaborate the reference§ made to the Russian tradition.

The development of structural analysis in Russia begins with the work of Daniel Bernouilli, who was invited there in 1725 by Catherine I, and was appointed to the Professorship of Mathematics in the Academy of St. Petersburg. Bernouilli was the first to obtain the differential equation of the deflection curve of a prismatic bar. He arrived at this by a consideration of the transverse vibration of the bar.‖ To his inspiration may also be credited the investigation of elastic problems by L. Euler (1701-83) who in 1733 succeeded Bernouilli in the Chair of Mathematics at St. Petersburg. Euler's most important contribution to the theory of structure is his discussion of the buckling of columns subjected to axial compressive forces. He treated not only the case in which the load is applied at the top of the column, but also the effect of the weight of the column. He was also the first to investigate the bending of bars originally curved. He introduced the assumption that the bending moment in this case is to be taken proportional to the difference between the original and the new curvature. This assumption was proved later by considering the nature of the elastic forces.

At the beginning of the Nineteenth Century the development of the theory of structures in Russia was greatly influenced by two Frenchmen, Lamé and

* This discussion (of the paper by H. M. Westergaard, M. Am. Soc. C. E., presented at the meeting of the Structural Division, Philadelphia, Pa., on October 6, 1926, and published in April, 1928, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Ann Arbor, Mich.

‡ Received by the Secretary, June 4, 1928.

§ *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 995.

‖ Commentarii, St. Petersburg Academy, 1741–43.

Clapeyron, who were professors in one of the oldest Russian engineering schools, "The Institute of the Engineers of Ways of Communications". This school was opened in 1807 and later had a prominent rôle in the development of engineering education in Russia. Professor Jouravski of this school was in charge of the building of the first railway bridges in Russia and was one of the first to develop methods of determining the forces in members of a truss. The Ministry of Ways of Communication was skeptical about his conclusion that in the Howe truss the forces in the verticals (bolts) produced by uniformly distributed load increase from the middle of the span to the supports. In order to demonstrate his theory, Jouravski used an interesting model. The verticals were replaced by strings and from the tones of these when plucked, the stress differences were readily shown.

He gave also the elementary theory of shearing stresses in beams, which is now found in all books on strength of materials.*

As concerns curved bars, H. Golovin was the first to give an exact theory of the bending of curved bars of a rectangular cross-section.† His work, published in Russian, remained unknown in Western Europe, and his solution was later rediscovered by L. Prandtl in Germany‡ and by Ribiére in France.§

The theory of elastic stability was advanced considerably by the work of Professor F. Jassinsky of the previously mentioned engineering school. He developed‖ the theory of the buckling of bars having elastic lateral supports and applied this theory to the investigation of the stability of compressed chords of through span bridges. To Jassinsky engineers are indebted also for the correct solution of the problem of the buckling of bars compressed beyond the yield point. His books on the mathematical theory of elasticity and on the theory of structures also had a great influence on the development of the teaching of these sciences in Russian engineering schools.

Among the Russian engineers of the beginning of the Twentieth Century may be mentioned A. N. Kriloff, who made considerable progress in the theory of vibration of bridges and ships¶ and I. G. Boobnoff who became prominent because of his work in the theory of the structure of ships.** He gave a very complete treatment of the bending of rectangular plates submitted simultaneously to hydrostatic pressure and to tension in the middle surface. Boobnoff was also the first to give an accurate solution of the problem of bending in rectangular slabs with built-in edges.

ALFRED D. FLINN,†† M. AM. SOC. C. E. (by letter).‡‡—Professor Westergaard's condensed and well co-ordinated summary of the successive steps along the several important lines of structural analysis, should be stimulative and

* *Annales* des Ponts et Chaussées, 1856, p. 328.

† *Bulletin*, Technological Inst., St. Petersburg, 1881.

‡ See A. Föppl, "Technische Mechanik," Vol. 5, 1907, p. 72.

§ *Comptes Rendus*, T. 108, 1889.

‖ "Theory of Buckling," St. Petersburg, 1892.

¶ *Mathematische Annalen*, Vol. 61 (1905).

** "The Strength of Ships," Vols. I and II, St. Petersburg, 1913.

†† Secy., United Eng. Soc., and Director, Eng. Foundation, New York, N. Y.

‡‡ Received by the Secretary, May 1, 1928.

helpful to further progress. It is a convenient and interesting historic narrative and annotated bibliography; but it is more. Professor Westergaard has produced an orderly series of skillful, critical, and lucid analyses of the contributions of the several distinguished workers in the field, who have made modern methods of structural analysis feasible. This summary illuminates the subject, makes relationships apparent, and indicates need for further researches.

The paper also emphasizes the facts that modern engineering is truly a learned profession and that it demands a rigorous educational preparation. That much vaunted "horse sense", if untutored, is no longer safe or sufficient equipment for the design of present-day structures to which human lives, limbs, and fortunes are committed. Adequate preparation for the practice of such engineering demands, also, better teaching than has been offered by some engineering colleges in years gone by.

JACOB FELD,* ASSOC. M. AM. SOC. C. E. (by letter).†—In summarizing, in such compact form, the original contributions to the science of structural analysis, the author has presented a paper which ranks in value far above the usual historical scientific paper. Although no attempt was made to explain the methods of attack of the men whose contributions are mentioned, the results as applicable to structural analysis as well as the effect of one man's work upon succeeding generations are clearly outlined. In spite of the enormous amount of historical data, the paper is written in a style which makes interesting reading. The addition of footnote references to the more important contributions makes this paper quite valuable as an outline for the research student in this field. It is to be hoped that it will be given widespread distribution among students and research workers.

The paper deals entirely with theoretical analysis as applied to structural design. It would be interesting to have a parallel discussion of the experimental work contributed by a great number of the men whose names are mentioned in the author's discussion, as well as by others, which has been used to check the theories proposed and to put the study on a scientific basis.

The discussion deals chiefly with statics, that portion of structural analysis which is used mostly by the membership of the Society. However, a great number of the contributions mentioned are really only special cases of the general theoretical discussions of dynamic principles. Considerable simplification has resulted in the very complicated equations for the general solutions of most dynamic problems by the use of vector notation and vector processes. This is merely a method of reducing the mechanical labor in mathematical derivation or a new method of placing certain ideas in symbolic form. The resulting vector mechanics has been applied in the mechanical and electrical fields as well as in the aerodynamic field. However, the recent expansion of structural design to the work required in airplane and airship production practically necessitates the inclusion of the general dynamic principles in the list of contributions to the field of structural analysis. The

* Cons. Engr., New York, N. Y.

† Received by the Secretary, July 5, 1928.

application of stress analysis for non-homogeneous bodies, such as reinforced concrete, has found a fertile field in the design of airships of various combinations of rigid keels and non-rigid envelopes. It is found that a great number of very theoretical contributions in stress analysis are quite applicable to the actual design of such structures.

Although separated in the text, the work in elastic stress by Cauchy and by Lamé are complementary, and together they give the general solution of the magnitude (by Cauchy) and of the direction (by Lamé) of stress. The results, analytically, are quite complicated; considerable simplification has been introduced by Foeppl by the use of vector notation.

The author has omitted a fairly important contribution to the subject of elasticity based on the so-called energy function. On this basis, Kirchhoff and Helmholz independently showed that an isotropic elastic body can be defined completely by two elastic constants, the modulus of shear or rigidity, and the bulk modulus of elasticity. Experimentally, the latter constant was studied by Karman.* His work on small cylinders subjected simultaneously to hoop and longitudinal pressures supplements his work on columns of ductile materials mentioned by the author.† Following the same idea of the energy function, Mr. S. E. Slocum has derived general formulas for the shearing deflection of beams of arbitrary, variable, or constant cross-section and also a general formula for the torsional deflection.‡ Most of the relations in indeterminate structures can easily be deduced from the idea of the energy function.

The writer recently presented a paper on the subject of "The History of the Development of Lateral Earth Pressure",§ and it is quite surprising to find how much repetition there is in names of contributors in the field of structural analysis and in that of soil mechanics. Both subjects seem to start with the same man, Coulomb, although previous work by a great number of men had paved the way. Just as Navier, Cauchy, and Poisson founded the theory of elasticity, Navier in his generalization of Coulomb's theory in 1826 to the cases of surcharged walls and also to the cases of soils containing cohesive as well as frictional resistances, and Cauchy and Poisson (whose works were the bases of the earth pressure theory by Maurice Levy, St. Venant, and Boussinesq), can be classed as the founders of the present two schools of soil study.

It is seldom realized how much progress in scientific study has been lost because of poor translation or the inclusion of the translator's own ideas into a book which bears the name of the original author. As a result, discrepancies will often be found in the reported contributions of certain men to any one subject, and it is quite important that the original references be consulted for a true evaluation. In this respect, Professor Westergaard's paper is especially good and, therefore, reflects considerable credit upon its author.

* *Zeitschrift des Vereines deutscher Ingenieure*, Vol. LV, 1911, pp. 1749–1757.

† *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1006.

‡ *Journal*, Franklin Inst., April, 1911, and July, 1912.

§ *Proceedings*, Brooklyn Engrs. Club, January, 1928, pp. 61–104.

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# PAPERS AND DISCUSSIONS



## LOAD DISTRIBUTION IN HIGH ARCH DAMS
### Discussion*

By Messrs. Paul Bauman and A. Floris.

Paul Bauman,† M. Am. Soc. C. E. (by letter).‡—This paper is of particular value as it represents the latest results of arch dam research from all over the world. If properly applied, this method of analysis will make possible the determination of maximum stresses with sufficient accuracy to satisfy the most conservative requirements of engineering design. These requirements pertain to engineering practice and must be kept apart from purely scientific requirements as long as dam designing is done by engineers and not by physicists.

The elastic behavior of an isotropic substance is fully described by the modulus of elasticity, $E$, and the coefficient of transverse contraction, $m$ (Poisson's ratio). The stresses in such a substance may be determined by the commonly known formulas. For anisotropic substances obeying Hooke's law, however, $E$ and $m$ must be replaced by twenty-one different constants which will quickly make the derivation of respective formulas impossible for practical use. What it would mean to make a theoretically exact analysis of a body of concrete, such as an arch dam, may be realized if consideration is given to the fact that concrete is not only anisotropic, but that it misbehaves with regard to Hooke's law. The dam designer, therefore, must be thankful for simple formulas, provided they are on the side of economy and safety. The stresses thereby determined in various points of the structure must then be considered as average stresses of a certain zone around these points.

The tests on the Stevenson Creek Dam have shown that the elastic theory holds good for relatively thin arches, which is a corroboration of many tests on bridge arches previously made in the United States and abroad; but the Stevenson Creek Dam is not representative of a high arch dam, and neither is the stress distribution as shown by the tests. The writer believes that the flow or plasticity of concrete in a high, massive arch dam of considerable

* Discussion on the paper by R. A. Sutherland, Assoc. M. Am. Soc. C. E., continued from September, 1928, *Proceedings*.

† Designing Engr., Quinton, Code & Hill, Los Angeles, Calif.

‡ Received by the Secretary, July 17, 1928.

volume alone will materially affect the load and the stress distribution. This will be a favorable influence due to its equalizing tendency.

The assumption of unyielding arch abutments is justifiable if the rock that forms the abutments is solid and is at least as strong as the concrete in the arch. It would not be in accordance with good engineering judgment to build an arch dam at a site not conforming to this condition. Even a gravity dam with sufficient curvature to permit arch action would be an unwise selection because the arch pressure of the top part is at times likely to be much in excess of the bottom pressure of the cantilever of maximum height. Consequently, a displacement of the arch abutments—with its well-known destructive effect on a hingeless non-reinforced arch—is quite possible.

The designer is often asked to prepare a preliminary design of an arch dam for the purpose of a cost estimate within such a short time that an accurate analysis is out of the question. It is then necessary to use a short cut which leads to both a safe and an economical result. The time element enters into the proposition beginning with the design! The number of sections for which the deflections are analyzed must then be reduced to a minimum, possibly to sections through the center and the quarter-points, or even to just one section through the line of maximum height. In the case of extremely short notice the writer has found that in designing a dam, as much as 300 ft. in height for arch action only, allowing an initial stress of 400 lb. per sq. in. at the crown and 365 lb. per sq. in. at the springing line, the volume of concrete will compare favorably with that obtained through accurate design for dam sites of ordinary, that is, parabolic, shape. The tensile stresses in the vertical beam which may be disclosed by an accurate design should never cause much concern inasmuch as the reinforcing steel required to take care of them will seldom exceed 1% of the cost. An investigation of the Sawpit Canyon Dam of the Los Angeles County Flood Control System, for example, has shown that the tensile stresses in the vertical beam due to simultaneous influence of full water load, shrinkage, drop in temperature, and elastic yield of abutments, could have been safely provided for with $1 700 worth of reinforcing steel (less than 0.3% of the cost). Since the introduction of reinforced concrete, the fear of tensile stresses is unwarranted.

The writer has often used the ellipse of elasticity both in bridge and dam design and, therefore, is pleased to see it used by the author. While the equations for elastic thrust and deflections due to uniform load are not simpler than those obtained by the work method, the elastic ellipse affords an excellent analysis of thrusts and deflections due to non-uniform and non-symmetric loads. In the case of a circular arch under uniform load (see Fig. 19), assume that the arch encastré at the right end and free at the left, is perfectly rigid. Obviously, then, the only condition for equilibrium is that the tangential reactions to the center line of the arch at each end, if combined with the resultant of the load, must form a closed triangle. Starting from the free end and letting one arch element after the other become elastic, the theory of the ellipse of elasticity shows that each element will, under the influence of the resultant, $N$, of all the forces acting at a distance, $S$, from its center of gravity, undergo an angular displacement: $d\,(\Delta\,\delta) = N\,S\,dg$. This angular

displacement or rotation takes place about the anti-pole of the force, $N$, with respect to the ellipse of elasticity of the element. These elemental rotations may be considered as weights acting at the anti-pole of each element. If, therefore, the whole arch has become elastic, its angular displacement must be the sum of all the elemental displacements and must take place about the center of gravity of the elemental rotations acting as weights.

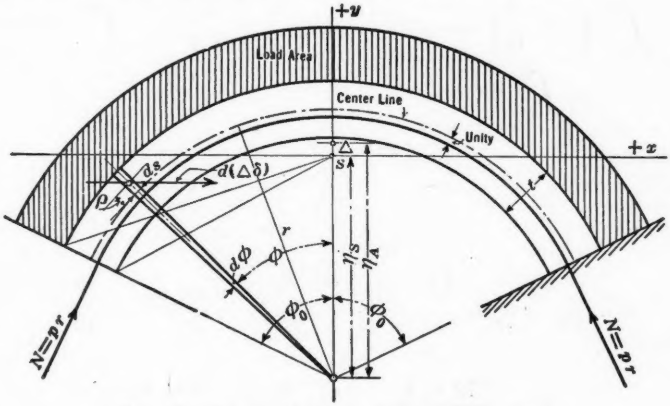


FIG. 19.—CIRCULAR ARCH, UNIFORMLY LOADED.

The supplementary force, $\Delta H$, necessary to establish equilibrium of the elastic arch, may be found from the conditional equation:

$$\Delta H \, f \int_0^{\phi_0} d\,g = \int_0^{\phi_0} d\,(\Delta\,\delta)$$

or,

$$\Delta H = \frac{\int_0^{\phi_0} N\,S\,d\,g}{f \int_0^{\phi_0} d\,g} \quad \text{.......................(92)}$$

in which, $f$ is the normal distance of $\Delta H$ from the elastic center of the whole arch.

For a circular arch under uniform load the line of pressure coincides with the center line and $S = 0$. To make Equation (92) applicable it is necessary to move the line of pressure concentrically up or down by a constant distance, $S$, which, if made unity, at once disappears from the equation.

Therefore,

$$\Delta H = \frac{N \int_0^{\phi_0} d\,g}{f \int_0^{\phi_0} d\,g} = \frac{N}{f} \quad \text{....................(93)}$$

Using the author's notation:

$$N = p\,r$$

$$f = \frac{i_x^{\,2}}{a} = \frac{I_x}{a \int_0^{\phi_0} d\,g}$$

$$a = \eta_A - \eta_S \text{ (Fig. 19)}$$

$$\eta_S = \frac{r \sin \phi_0}{\phi_0}$$

$$d\,(\Delta\,\delta) = r\,p \times 1\,d\,g$$

$$\eta_A = \frac{\int_0^{\phi_0} \left(r + \frac{t^2}{12}\right) \cos \phi\, d\,(\Delta\,\delta)}{\int_0^{\phi_0} d\,(\Delta\,\delta)} = \frac{\frac{\left(r + \frac{t^2}{12}\right) r^2\,p}{E\,I} \int_0^{\phi_0} \cos \phi\, d\,\phi}{\frac{r^2\,p}{E\,I} \int_0^{\phi_0} d\,\phi}$$

$$= \frac{\left(r + \frac{t^2}{12}\right) \sin \phi_0}{\phi_0}$$

Therefore,

$$a = \eta_A - \eta_S = \frac{\left(r + \frac{t^2}{12}\right) \sin \phi_0}{\phi_0} - \frac{r \sin \phi_0}{\phi_0} = \frac{t^2\,l}{12\,L}$$

and,

$$f = \frac{12\,I_x\,L}{t^2\,l \int_0^{\phi_0} d\,g} = \frac{I_x\,E\,t^3}{t^2\,l} = \frac{6\,r^2\,K_0}{t^2}$$

Introducing these values,

$$\Delta\,H = \frac{p\,t^2}{6\,r\,K_0} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(94)}$$

which is identical with Equation (24).*

The real value of the foregoing theorem lies in its application to arches under non-uniform, non-symmetric loads. $\Delta\,H$, therefore, no longer acts along the $x$-axis, but lies obliquely to it, producing a component in both the $x$ and the $y$-axes. A graphical analysis will then give quickest results. Such an analysis is shown in Fig. 20. The arch is divided into elements of equal and finite length and their centers of gravity, elastic weights, $\Delta\,g$, $\rho$ and $\rho'$ are determined. The center of gravity of the elastic weights of the whole arch is then determined and through it, the $x$ and $y$-axes as principal axes of inertia of the arch. Major and minor axes of the ellipse of elasticity of the whole arch are then computed and plotted, and the ellipse is drawn, thereby establishing a scale of the elastic properties of the arch which are constant for one and the same arch and, therefore, are not affected by any load on the arch.

The elemental division lines are now extended into the load area, the elemental loads are computed and combined in a convenient scale with an arbitrary pole to a force polygon. The corresponding funicular polygon is then drawn in any position, observing, however, for the sake of simplicity that it does not intersect the arch axis. After determining the anti-poles of the forces (line of pressure), with respect to the elemental ellipses of elasticity, and letting the elemental rotations, $\Delta\,\delta = N\,s\,\Delta\,g$, act at the anti-poles as weights

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1048.

parallel to both the $x$ and the $y$-axes, their center of gravity, $A$, may be found. As $A$, also, is the anti-pole of $\Delta H$ with respect to the ellipse of elasticity of the whole arch, the position of $\Delta H$ is at once determined and its magnitude is:

$$\Delta H_Y = \frac{\Sigma \Delta \delta}{f \Sigma \Delta g} \qquad (95)$$

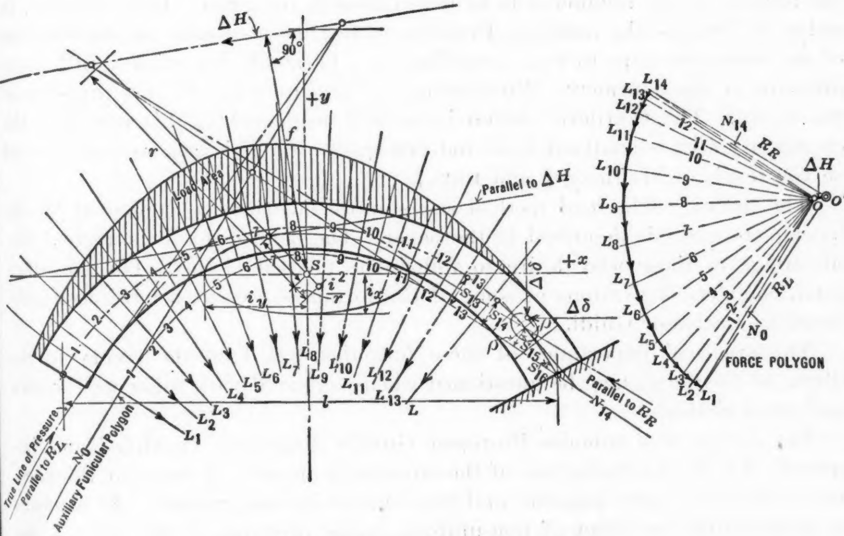


FIG. 20.—GRAPHICAL ANALYSIS OF ARCH.

By plotting $\Delta H$ from Pole $O$ negative, that is, contrary to its direction of action, the true pole, $O'$, is established. The two force polygons, the temporary and the true one, are affined with $\Delta H$ as their axis of affinity. The true line of pressure, therefore, is found by remembering that corresponding rays (forces) intersect on the axis of affinity. For a change of temperature of $\pm T°$ Fahr.:

$$\Delta H_T = \frac{\pm \Delta T \alpha E l}{\Sigma \Delta g i_x^2} \qquad (96)$$

For relatively thin arches it is permissible to use $I_x = \Sigma y^2 \Delta g$ and $I_y = \Sigma x^2 \Delta g$, whereby a material reduction in computation is effected. This approximation gives somewhat smaller values for $I_x$ and $I_y$ and, therefore, is on the side of safety.

A. FLORIS,* ESQ. (by letter).†—The method of determining stresses in a dam by using the conception of the combined cantilever and arch action is comparatively well known. One investigation, dealing with the arch crown alone, dates back to 1889,‡ and several attempts have been made to determine the stresses in other points besides the crown. The author's method of analysis

* Los Angeles, Calif.

† Received by the Secretary, July 23, 1928.

‡ "On the Strains in Curved Masonry Dams," H. Vischer and L. Wagoner, *Transactions*, Technical Soc. of the Pacific Coast, December, 1889, p. 75.

was applied to the design of the La Jogne Dam* in Switzerland, and to several other dams† of the U. S. Bureau of Reclamation.

Professor C. Guidi also gives several sets of equations‡ for the determination of stresses at various points of the arch, using the cantilever and arch action theory. He states, however, that although the solution of these equations presents no special difficulty, they are of little practical value because the number of the unknowns to be determined is too great. Consequently, in order to simplify the analysis, Professor Guidi has assumed the distribution of the water pressure to vary according to a parabolic law with a full water pressure at the abutments, diminishing to three-quarters of this pressure at the crown. The cantilever action is entirely neglected in this case and the elementary arch is analyzed as an independent unit. This assumption is based on the result of his theory§ and tests.‡

The author's trial-load method seems quite laborious as indicated by the incomplete analysis described in the paper. The lack of detail is a great disadvantage to those who desire to follow the method closely. On the other hand, complete derivations of some of the formulas presented have been published by Professor Guidi.

The practical importance of these formulas, based on the theory of the ellipse of elasticity, may be questioned when compared with other well known analytical methods.

The author also submits Professor Guidi's diagrams‖ (modified and improved), for the determination of the stresses in an arch of constant thickness due to uniform water pressure and to a change in temperature. As a means of determining the effect of non-uniform water pressure on the stresses, the author has extended Professor Guidi's influence lines¶ to cover some special cases. This is perhaps the most valuable part of the paper, which will serve, somewhat, to familiarize American engineers with Professor Guidi's scientific investigations on the design of arch dams.

* "Etude sur les barrages arqués," A. Stucky, *Bulletin technique* de la Suisse romande, 1922.

† "Analysis of Arch Dams by the Trial Load Method," C. H. Howell, M. Am. Soc. C. E., and the late A. C. Jaquith, Esq., *Proceedings*, Am. Soc. C. E., Papers and Discussions, January, 1928, p. 61.

‡ "Studi sperimentali su costruzioni in cemento armato," C. Guidi, Estratto dal rendiconto dell' XI Riunione dell' Associazione Italiana per gli studi sui materiali da costruzione, Turin, 1926, p. 18; or its French translation, "Etudes expérimentales sur des constructions en béton armé," A. Paris, *Bulletin technique* de la Suisse romande, 1927, p. 19; "Esperienze termiche su di una diga a volta," C. Guidi, Estratto dagli Atti della Reale Accadimia delle Scienze di Torino, Vol. LXII, 1927.

§ "Statica delle dighe per laghi artificiali," C. Guidi, Turin, 1921, p. 71; and "Sulle dighe ad arco," C. Guidi, *Estratto da L'Ingegnere*, Vol. I, August, 1927.

‖ "Statica delle dighe per laghi artificiali," C. Guidi, Turin, 1922, Appendix, pp. 6 to 7.

¶ *Loc. cit.*, p. 70.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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# PAPERS AND DISCUSSIONS



# THE DESIGN OF TALL BUILDING FRAMES TO RESIST WIND

## Discussion*

BY MESSRS. DAVID C. COYLE, ALBERT SMITH, ROBINS FLEMING, AND EVERETT E. EBLING.

DAVID C. COYLE,† M. AM. SOC. C. E. (by letter).‡—The theory involved in this method has one point which, the writer believes, needs further elucidation. Equations (10),§ (12),§ (13)§ and (15)‖ are written as if $R$ were a constant for columns above and below the floor. This assumption can be shown to involve a third condition in addition to those given by the authors§ namely, that the $K$ values of the columns must be proportional to their lengths. This condition does not occur generally in the first story, which is apt to be higher than the second. The method also becomes questionable where a two-story arcade divides the building into sections of different story heights. Subject to these limitations, this method gives more accurate results than the usual forms, at a considerably less cost of labor than the straight use of slope deflections.

There is the general question of the whole underlying assumption of the more exact theories, which ought to be fully discussed before designers come to any final conclusion as to method. In the first place, the steel is required to hold the building against wind while the walls are under construction, but it is a question whether, after the masonry is in place, its effect on the stiffness of the building is appreciable; that is, whether doubling the amount of wind-bracing makes an appreciable change in the period and amplitude of vibrations. This doubt does not of course apply to $x$-bracing. Another question is whether the ultimate strength is of any consequence, since in the hurricane at Miami, Fla., no building with well-constructed masonry was damaged. Again, granting that it is desirable to use wind-bracing as a safety measure, the real point where judgment and experience are applied in practice

* This discussion (of the paper by Albert Ward Ross, Jr., Jun. Am. Soc. C. E., and Clyde T. Morris, M. Am. Soc. C. E., presented at the meeting of the Structural Division Columbus, Ohio, October 13, 1927, and published in May, 1928, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Designing Engr., Gunvald Aus Co., New York, N. Y.

‡ Received by the Secretary, January 24, 1928.

§ *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1411.

‖ *Loc. cit.*, p. 1412.

is not so much in the choice of theory as in the choice of unit load. Shall 10 lb., or 35 lb., per sq. ft. be carried by the steel? The writer has designed a large building for 10 lb. and a much smaller one for 30 lb.; and has felt more uneasy as to the latter, in respect of possible complaints from tenants about vibration. After all, if one is deciding between unit loads in such a wide range, how important is a 30% error in the mathematical theory?

A still more fundamental question is: What is meant by an adequate design for wind resistance? Any decent design will stand up in a hurricane, as the Miami storm demonstrated; but sometimes a tenant moves out because the vibration in a storm makes him uneasy. Perhaps the real problem is not safety, but the avoidance of those periods and amplitudes of vibration which are perceptible to human beings. If this is the case, then unit loads and stresses are all purely nominal; they are expressions of a designer's judgment as to the probable motions which may occur in the particular building in question and how annoying these motions might be to people in an office, or lying in bed in the dark, as the case may be. Designers ought, perhaps, to take more into account the shape of the building, even to the extent of judging that if it is too slender it can under no circumstances be designed so as to feel right in a storm, although it may easily be designed so as to be safe.

Obviously, what the writer is proposing is that structural engineers shall be led to the realization that wind-bracing is not altogether a science, but in some ways an art—the adjustment of a material structure to the somewhat intangible requirements of the human nervous system. This is an extreme statement, but it expresses a fact that should be kept in mind. However rough or however elaborate the method, what the structural engineers shall be judged by is, in practically all cases, the "feel" of the structure and nothing else.

The fact that in practice wind-bracing is so largely a matter of judgment makes it particularly valuable to have thorough studies made of the subject, such as that of Professor Morris and Mr. Ross. Any one who has thoroughly studied the Ross method and has understood it, has thereby definitely improved his capacity to decide correctly the points where judgment is required. He has also improved his understanding of the scope and limitations of the mathematical theories in common use. The writer is convinced, therefore, that this paper will be a permanently valuable addition to the literature of the subject, in spite of all reservations as to the limitations of mathematical methods in general.

Albert Smith,* M. Am. Soc. C. E.—In high and narrow buildings of irregular shape no part of the design is so difficult as that of the wind-bracing. In office buildings, columns are usually lined up, but in high apartment buildings they are likely to be placed haphazard, and the task of providing a path for the wind forces is made very perplexing indeed.

The most difficult points are found to be: First, apportioning the proper values of wind shear to the various bents according to stiffness; second, appraising truly the resistance offered by beams of varying depth in a given

* (Smith & Brown), Chicago, Ill.

bent; and, third, avoiding a tendency to impose on any given bent a shear and moment which will develop an uplift greater than a conservative proportion of the gravity load on the column.

In computing the relative stiffness of bents it is necessary to assign approximate values to the length of columns and girders. Where moment connections of different types are used, this becomes very difficult. For instance, if some connections are riveted brackets, obviously the end rivets on the brackets will produce bending of the angle or I-beam flanges. On the other hand, for the I-beam lug connections which are very positive, the only ambiguity in length is due to width of column and depth of beams. An angle lug connection and an I-beam lug connection both involve an ambiguity in the value of $I$ to be used in determining stiffness, although the assumption that lateral movement is equal at all points of any floor would indicate that if any bracket connections are present, the lug connections cannot be over-stressed.

It is very difficult to make connections to columns, the webs of which are parallel to the wall. Usually, where the wall is a major line of bracing the moment of inertia of the columns about an axis parallel to the web is not sufficient to permit placing the webs normal to the wall.

The development of multiple punching and drilling makes the detailers and fabricators very eager to keep typical gauge spacing. This makes it difficult to secure such gauges on angle brackets that the rivets can be counted at full value.

Eccentricity of wall-spandrel connections to columns should be carefully considered in places where the moment in the column produced by wind is approximately equal to the allowable strength of the column. The speaker considers that attachment to one flange parallel to the wall enables him to count little more than the moment of inertia of that one flange. If, on the other hand, the connection to a column with webs parallel to the wall, is so made that the outer flanges are securely tied together, he assumes that the limit of strength, up to the full $I$ of the column, is determined by the floor attachment of the spandrel girders and the torsional stiffness of the spandrel connection.

Many unsymmetrical conditions make the application the most simplified of the authors' formula inapplicable to many buildings. The attempt to make a rational solution of the combined action of many bents in many stories would involve a tremendous amount of labor and would not be exact.

The design of the wind-bracing of most buildings has to be done at great speed. The speaker thinks it better therefore to use a conventional assumption in regard to points of contraflexure and distribution of shears, modified roughly to allow for a symmetry and variations in stiffness, than to attempt greater refinement of calculation.

ROBINS FLEMING,* ESQ. (by letter).†—A broad classification of methods for designing tall building frames to resist wind is:

1.—Theoretical methods (often called "exact" methods); and,

2.—Workable methods (often called "approximate" methods).

* Am. Bridge Co., New York, N. Y.

† Received by the Secretary, June 19, 1928.

A theoretical method is of but little use unless it is workable, that is, practical to follow. An ideal method would be based on sound theory and also, would be workable. Let it be emphasized that no method should be used for the sole reason that it can be worked quickly and with ease. Safety should always come first, regardless of the time it takes to provide for it. Lack of time to make a better one is not a valid excuse for an engineer to offer for a poor design.

The so-called exact methods have one fault in common—they are not workable. The oldest of them is that of Mr. Ernst F. Jonson,* and the least workable of any method ever presented is, probably, that of Cyrus A. Melick,† M. Am. Soc. C. E. Albert Smith, M. Am. Soc. C. E., presented his "method of least work" in 1915.‡ The Wilson and Maney slope-deflection method is simpler than any of the preceding ones, although the number of equations and of unknowns mounts up rapidly with the number of stories. Messrs. Ross and Morris found that thirty-two of these were required for the lower eight stories of the building they considered.

The tall building frames—the skyscrapers—in the United States are counted by the thousands, and the number is steadily increasing. More than one hundred applications for permits to build structures of twelve stories or more in height have been filed in New York City alone each year since 1918. It is doubtful if "exact" methods of calculating wind stresses have been used in connection with a dozen of all the high buildings thus far erected. The writer personally knows of but three.

To come now to the paper under discussion—from a theoretical standpoint, it is excellent. The assumptions and the method of calculating stresses are sane although it is pertinent to ask whether the actual conditions will agree with these assumptions in the determination of stresses. They may do so more nearly than by other methods. The writer, however, has no intention to pass judgment on the technical merits of the paper—this will be left to others; he will only ask, "Will the proposed method be used to any extent in the design of future high buildings?"

To answer frankly, the writer believes that the method will be used but little. The reasons for this opinion are obvious from the section entitled "Routine Outline to Be Used in Design."§ Paragraph 5 stipulates that the columns be proportioned to carry the live and dead load, together with bending and direct stresses, as previously calculated by Paragraphs 2, 3, and 4. Paragraph 6 calls for a "key girder" to be designed. The next step is to proportion the other girders by their relative $K$'s and check to see that the allowed unit stresses are not exceeded for live and dead loads.

The preliminary design is now completed "and it will usually be found to differ materially from theoretical proportions". The revision of stresses is stated in Paragraphs 8 to 17. Space conservation forbids reproducing them, but the steps to be taken involve more time than can be given by the great

* *Transactions*, Am. Soc. C. E., Vol. LV (1905), p. 413.

† *Bulletin No. 8*, Coll. of Eng., Ohio State Univ. (1912).

‡ *Journal*, Western Soc. of Engrs., Vol. 20 (1915).

§ *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1417.

majority of those called on to design the steelwork of high buildings. This may be lamented, but it is a fact. The statement* that "the new method of calculation requires little more time than is required for any of the approximate methods in common use", will be disputed, if not flatly contradicted. The further statement that the entire set of calculations for a Wilson and Maney 20-story bent was made by the writers of the paper in six hours will not carry great weight. While this may be done by the occupant of a Professor's chair, it seldom can be done elsewhere.

Time alone will show to what extent the method will be used. It may be interesting to note that while this discussion was being written, an engineer was designing wind bracing in accordance with the authors' method. He had proceeded as far as the fourth paragraph of the "Routine Outline" when, seeing that he had thirteen more steps to take, he switched over to an approximate method.

As stated by the authors, various methods for determining wind stresses in steel frame buildings have been presented by the writer from time to time. The latest of these and the one that represents his best thought is given in the English periodical, *Engineering*, in which he has confined himself to two methods. One of these may be called the "cantilever method" for determining wind stresses in the gusset-plate type of high buildings.† This is a slight modification of that presented by A. C. Wilson,‡ M. Am. Soc. C. E., and is the Method No. 1 which the authors believe "may give results which are seriously in error".

The Bethlehem Steel Company follows this method in the section, "Wind Stresses in Tall Buildings", of its handbook entitled "Bethlehem Structural Shapes". It is based on the following assumptions: (1) All columns in a given story have equal sections, and the direct stresses in the columns due to wind are proportioned to the distances of the columns from the neutral axis of the bent; (2) the point of contraflexure of each column is at the mid-height of the story, and the point of contraflexure of each girder is at its mid-length; (3) the resultant of the wind pressure acting between two successive points of contraflexure is applied at the intersection of column and girder; (4) the joints are perfectly rigid. It will be noted that Assumption (1) will cause the bending moments in the floor girders to vary.

A second method may be called the "portal method." This is one of the methods declared by Wilson and Maney to be "so inaccurate that they should never be used". A transverse bent is regarded as a series of independent portals. The total horizontal shear at any story is divided equally between the number of aisles. An outer column thus takes only one-half the shear of an interior column. Assumption (1) of the "cantilever method" would, therefore, have all direct stresses due to wind carried by the exterior columns. Assumptions (2), (3), and (4) remain unchanged. It will be noted that the bending moments, due to wind, for all girders on the same floor of any transverse bent,

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1433.

† "Wind Stresses in Many-Storied Buildings," by Robins Fleming, *Engineering*, May 25, 1928, pp. 625–628.

‡ "Wind Bracing with Knee-Braces or Gusset-Plates," A. C. Wilson, *Engineering Record*, September 5, 1908, p. 272.

are alike. This is an ideal condition for the detailer and the shop. The designer finds this method very simple, and his work may be easily checked.*

At one time, after carefully designing a 20-story building according to the "cantilever method," the writer found that the detailer had made all end connections of girder beams in the same floor alike. In detailing, the sum of the girder moments in the floor of a transverse bent had been divided by the number of girders and the result had been used as the average bending moment. Incidentally, one advocate of the "cantilever method" does this regularly. Of course, this tends to throw direct stresses to the exterior columns, as in the "portal method."

After the Florida hurricane, it was found that the floor construction of high buildings was little damaged. Hardly a crack was seen in stone or cinder concrete floors where windows were blown in and walls considerably shaken. This would seem to indicate that horizontal wind pressure was distributed by the floor to all columns. One engineer, with a wide experience in the design of the steel frame of high buildings, determines the total wind shear in a story and divides it among the columns according to their respective moments of inertia.

The writer believes that either of the foregoing methods, followed consistently with such modifications as may be needed by the case under consideration, will give a safe structure without undue waste of material. He would not for a moment consider such a building poorly designed. He is at present inclined to give preference to the "portal method".

Monumental structures and those embodying special or unusual features are in a class by themselves and should be considered separately. The American Insurance Union Building belongs to such a class. The owners are to be congratulated in having the wind stresses determined under the direction of such able engineers.

EVERETT E. EBLING,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The writer spent considerable time recently in applying the slope-deflection method to building frames.§ One part of the work was the solution of three different frames for lateral loads. Frame A was symmetrical, two stories high and two bays wide; Frame B was nearly symmetrical, three stories high and four bays wide; and Frame C was very unsymmetrical, three stories high and four bays wide.

The method proposed by the authors‖ was applied to these frames. The results were very nearly those obtained by the exact method (even in the very unsymmetrical frame) except for the basement columns and first floor girders. These results were uniformly too large. The differences were considerable for Frames B and C. The reason for these discrepancies is that the actual points of contraflexure in the basement columns lie above the mid-height.

* The method is given by Milo S. Ketchum, M. Am. Soc. C. E., in his "Steel Mill Buildings" and by other writers.

† Engr., Kalman Steel Co., Chicago, Ill.

‡ Received by the Secretary, August 13, 1928.

§ "Methods of Stress Analysis in Frames of Skeleton Type Office Buildings," presented as a thesis for the degree of Master of Science at Iowa State Coll., June, 1928.

‖ *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1415.

In Frame C, the actual point of contraflexure in some of the columns was above the upper quarter-point.

The points of contraflexure in the basement columns depend on the relative stiffness of columns and girders. In Fig. 8, the ratio of the $K$'s of the girder to the $K$'s of the column is plotted against the point of contraflexure. It might seem that there were not sufficient points to justify drawing a curve, but a little reasoning will show that for an infinitely small ratio there would be no point of contraflexure in the column; therefore, the curve will cross the vertical axis at 1.00. Similarly, for an infinitely large ratio, the girders would be rigid and the point of contraflexure would fall at the mid-height; therefore, the curve will be asymptotic to the horizontal axis at 0.50.

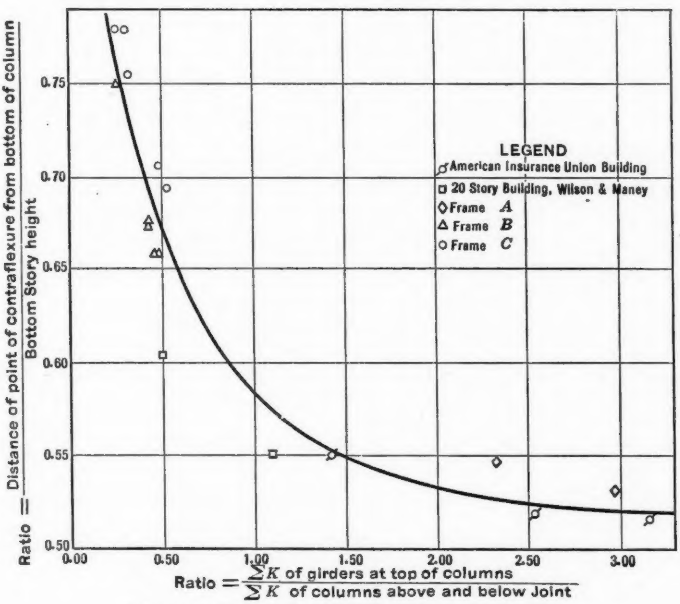


FIG. 8.—STUDY OF BOTTOM STORY COLUMNS.

Both the American Insurance Union Building and the twenty-story building analyzed by Wilson and Maney* are slender buildings. Consequently, the ratio of the $K$'s of their lower girder to the $K$'s of the column is large and the points of contraflexure fall near the mid-height of the columns. For this reason, the discrepancies in the authors' solutions are small. In a building having about the proportions of twenty stories high to eight bays wide, the ratio will fall on the upper part of the curve (Fig. 8) so that it would seem worth while to predict the point of contraflexure before calculating the first floor girder moments.

* *Bulletin No. 80*, Univ. of Illinois Eng. Experiment Station.

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# PAPERS AND DISCUSSIONS



## A PROPOSED FORMULA FOR COLUMNS

### Discussion*

By William R. Osgood, Assoc. M. Am. Soc. C. E.

William R. Osgood,† Assoc. M. Am. Soc. C. E. (by letter).‡—Equation (8)§ may be obtained on the assumption that the curve, *A C B*, (Fig. 1 (*b*)),‖ is a parabola, and that the curve, *A O B*, is, relative to *A C B*, a cosine curve of half wave length equal to $l$. The curve, *A O B*, is given by the equation,

$$y = \frac{4\,\Delta\,x^2}{l^2} + y_1 - y_1 \cos \frac{\pi}{l}\,x \quad \text{.................} (31)$$

If the bent axis, *A O B*, is assumed to be a cosine curve of half wave length equal to $l$,

$$y = (y_1 + \Delta)\left(1 - \cos \frac{\pi}{l}\,x\right) \quad \text{.................} (32)$$

the deflection, $y_1$, is obtained as,

$$y_1 = \frac{\dfrac{P\,l^2}{E\,I}\left(\dfrac{\Delta}{9.87} + \dfrac{e}{8}\right)}{1 - \dfrac{1}{9.87}\,\dfrac{P\,l^2}{E\,I}} \quad \text{.....................} (33)$$

It is not clear just what the advantages of the proposed formula are. As shown by the dotted-line curves in Fig. 7, the proposed formula for columns with pivoted ends can be approximated by the Johnson parabola and the Euler curve, which is tangent to the parabola; that is, for $\frac{l}{r} \leqq 118$, by the curve,

$$\frac{P}{A} = 14\,000 - \frac{1}{2}\left(\frac{l}{r}\right)^2$$

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* This discussion (of the paper by T. F. Hickerson, M. Am. Soc. C. E., published in May, 1928, *Proceedings*, but not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Asst. Prof., Structural Eng., Cornell Univ., Ithaca, N. Y.

‡ Received by the Secretary, May 14, 1928.

§ *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1438.

‖ *Loc. cit.*, p. 1436.

and, for $\frac{l}{r} > 118$, by the curve,

$$\frac{P}{A} = \frac{98\,000\,000}{\left(\frac{l}{r}\right)^2}$$

The approximation is excellent for values of $\frac{l}{r}$ between 20 and 120; and for larger values of $\frac{l}{r}$, the values of $\frac{P}{A}$ are somewhat smaller, corresponding (with $E = 30\,000\,000$ lb. per sq. in.) to a factor of safety of about 3 for centrally loaded columns. It should be remembered, also, that the proposed formula is based on the assumption of arbitrary values of $\Delta$ and $e$, and that, in general, a different formula will be obtained for another set of values. Little is known about the actual values of these quantities in a practical column.

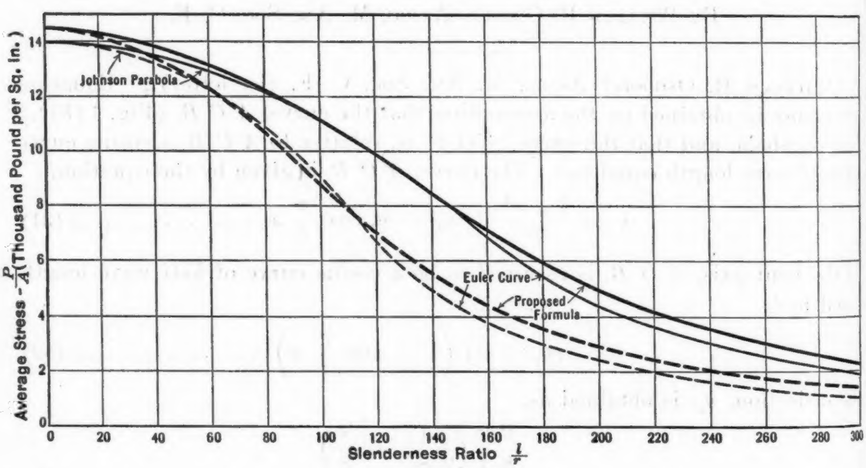


FIG. 7.

The Johnson formula and the Euler formula have the distinct advantage of being easy to use when it is a question of designing a column. The former may be transformed so as to read,*

$$A = \frac{P}{14\,000} + \frac{1}{28\,000}\,\frac{A}{r^2}\,l^2 \qquad (34)$$

The quantity, $\frac{A}{r^2}$, has the same value for all cross-sections which are geometrically similar, and it varies slowly for cross-sections which are nearly similar, for example, for rolled steel sections of any given approximate shape. Values of $\frac{A}{r^2}$ for a number of different cross-sections are given by A. Ostenfeld.† With $P$ and $l$ known and with the form of cross-section selected, it is only necessary

* "Teknisk Elasticitetslære," by A. Ostenfeld, Copenhagen, 1924, pp. 464 ff., or "Columns", by E. H. Salmon, Lond., 1921, p. 247.

† "Teknisk Elasticitetslære", p. 466.

to solve for $A$, to pick out the proper section, and possibly to check the value found for $A$, by substituting the correct value of $\frac{A}{r^2}$ if this value differs appreciably from the first value used. The second formula may be solved, of course, directly for $I$ (with $I = A\,r^2$):

$$I = \frac{P\,l^2}{98\,000\,000}$$

In the same way, the proposed formula for columns with restrained ends may be replaced advantageously by the formulas (see full-line curves in Fig. 7), for $\frac{l}{r} < 158$,

$$\frac{P}{A} = 14\,000 - \frac{9}{32}\left(\frac{l}{r}\right)^2$$

and, for $\frac{l}{r} \geqq 158$,

$$\frac{P}{A} = \frac{174\,000\,000}{\left(\frac{l}{r}\right)^2}$$

# AMERICAN SOCIETY OF CIVIL ENGINEERS

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## PAPERS AND DISCUSSIONS

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## THE STIFFNESS OF SUSPENSION BRIDGES

### Discussion*

By Messrs. Lloyd G. Frost and Hans H. Rode.

Lloyd G. Frost,† Assoc. M. Am. Soc. C. E. (by letter).‡—The method developed by the author should serve to advantage as a check on the established procedure; its mathematical concept is clear and ably expounded. The title, however, is misleading, giving as it does the impression that the subject matter deals with the stiffening system from the standpoint of comparative design, whereas it is found to be a purely mathematical treatise in which a method is developed by which calculations can be made in a "much shorter time than by the usual method."

This description suggests the possibility of applying this method to the preparation of cost estimates. In this connection the writer has recently had occasion to prepare complete and exhaustive estimates and investigations of the cost of a proposed suspension bridge over the Mississippi River at New Orleans, La. These investigations were extended to comparisons of fixed and rocker towers, eye-bar and wire cables, also the combination of the cables with a part of the top chord of the stiffening truss, and the variation of the depth of the stiffening truss after the method used by D. B. Steinman and William G. Grove, Members, Am. Soc. C. E., in the design of the Florianopolis Bridge,§ was reviewed as well.

Estimates were first based on the elastic theory and then revised by the deflection method. Finally, the results obtained were compared with estimates prepared from the data and curves presented by J. A. L. Waddell, M. Am. Soc. C. E.‖ The agreement was within 3%, which amounts to a practical check.

* This discussion (of the paper by S. Timoshenko, Esq., published in May, 1928, *Proceedings*, but not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chf. Bridge Designer, New Jersey State Highway Comm., New Brunswick, N. J.

‡ Received by the Secretary, June 4, 1928.

§ *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 707.

‖ *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 884.

The considerable labor involved in determining a satisfactory basis of estimate amounted virtually to a complete design as regards fundamentals, and the results finally obtained indicate that for all practical purposes of preliminary estimates, curves such as those prepared by Mr. Waddell are sufficiently accurate. It is well known that very few engineers are called upon to design suspension bridges; and then, in many cases, such work goes no further than preliminary estimates of cost.

Most engineers are inclined to view with a certain skepticism any mathematical "short-cuts" with which they are unfamiliar; and, if used, to check them thoroughly with methods they are accustomed to employ. The paper under discussion presupposes a facility with the calculus that, in the writer's opinion, a majority of able and practical designers do not have. The method developed in this paper is to some extent shorter than that ordinarily used in calculations of this nature; but the practical value of such shortening seems somewhat doubtful.

In the rare occasions that the engineer is called upon to design suspension bridgework, his first and in all probability his only necessity for quick methods of calculation will have to do with the preparation of preliminary estimates of cost. Once the work is in the hands of the designers for final development, the matter of time involved in design calculation becomes of small moment; and the calculation of bending moments and deflections in the truss constitutes a very small part of the work involved.

While perhaps of interest to the engineer who has a flair for abstract mathematical analysis, the writer believes this paper to be largely another leaf in the already imponderable tome of matter that serves but a small minority and is of little value to the busy engineer in the practice of his profession.

HANS H. RODE,* M. AM. SOC. C. E. (by letter).†—This paper, presented by an eminent authority in the field of applied mathematics, shows that trigonometric series can be utilized to good advantage in computations pertaining to suspension bridges. It is to be hoped that in the future such series will be as well known and as frequently used by civil engineers as they are now by the mechanical and electrical engineers.

The writer, however, cannot agree with the more specific procedure recommended in this paper. The author assumes that the additional horizontal component, $H$, from the live load, $p$, may be neglected in comparison with the horizontal component, $H_w$, from the dead load, $w$, and, on this basis, he finds a maximum deflection, $\mu_{max.} = 22$ in. for a concentrated load of $P = 100\,000$ lb. at the middle of the main span of the Manhattan Bridge.

This is correct enough according to the author's method, which amounts to assuming the cables carried over sheaves and counterweighted so as to maintain a constant stress in them, but it does not represent actual conditions in suspension bridges. If the assumption were correct, large bridges would deflect 30 or 40 ft., or more, and their stiffening trusses would break. Using

* Prof., Norges Tekniske Höiskole, Trondhjem, Norway.

† Received by the Secretary, June 18, 1928.

the author's numerical data, the correct value of $\mu_{max.}$ is found to be approximately 2 in.

In computing $\mu_{max.}$ the author has entirely ignored the condition that the horizontal projection of the cable must equal the distance between anchorages. Without this condition, in some form or other, the deflections and stresses cannot be determined.

The deductions in this paper are also based on Equation (4)* which is frequently encountered in books and discussions and seems to be universally accepted as correct—which it is not.

The writer has found that the exact differential equation is rather complicated, but that it assumes a simple form if the cable strain—that is, the elongation of the cable under live load stress or from temperature, is neglected. In that case, and assuming a parabolic cable curve, the equation will be:

$$E\,I\,\frac{d^4\,\mu}{d\,x^4} = p - \beta\,w + H_w\,(1+\beta)\,\frac{d}{d\,x}\left(\frac{d\,\mu}{d\,x}\sec^2\phi\right) = p - \beta\,w$$

$$+ H_w\,(1+\beta)\left[\frac{d^2\,\mu}{d\,x^2} + \frac{d^2\,\mu}{d\,x^2}\,t\,z^2\,\phi - \frac{d\,\mu}{d\,x}\,\frac{16\,f}{l^2}\,t\,z\,\phi\right]\,\ldots\ldots\ldots(23)$$

in which, $\phi$ = the slope angle of the cable curve at any point, and,

$$\sec^2\phi = 1 + t\,z^2\,\phi = a + b\,x + c\,x^2$$

Equations (4) and (23) differ only in the last term on the right-hand side.

Ordinarily, Equation (4) constitutes a good approximation. This is particularly true for cables with a small sag ratio, although the effect depends largely also on the type of loading. It should be borne in mind, however, that the errors resulting from the use of Equation (4) are, in some cases, not altogether negligible.

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1466.

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# PAPERS AND DISCUSSIONS



## IMAGINATION IN CITY PLANNING

### Discussion*

By Messrs. R. D. N. Simham and M. W. Weir.

R. D. N. Simham,† Assoc. M. Am. Soc. C. E. (by letter).‡—The writer agrees with the author that civic orderliness and harmony constitute the main features of city planning, but prefers to call the latter the "soul" of city planning. Without this soul of harmony in a city, life would be intolerable and, sooner or later, decay would overtake it. In truth, "harmony" is the soul of the eternal life of the Universe and Nature; and a city is, after all, a man-made model or plan of the habitable universe, in some limited way. Following, therefore, the fundamental laws of Nature, a city plan must fit in perfectly with the past, strictly conform to the present, and freely adapt itself to the future.

A great Indian saint and author of a famous treatise on the Indian systems of planning kingdoms is responsible for the statement: "First plan the city (or town or village or hamlet) and then only lay out the buildings and houses; any violation of this rule portends and brings evil." In India, the temple must always exist and must stand in the center of a city, as the heart of the community, in spite of any advance in the status of industries and commerce.

Garden city planning is a very ancient ideal to India. All these traditional planning systems uniformly insist on incorporating in the city plans the ideals of beauty, convenience, health, sanitation, and other amenities. It is stated, therefore, in ancient city planning treatises, that "trees shall be planted first and the dwellings erected thereafter; otherwise they will not be pleasant, and will not look graceful and seemly". It is also enjoined in "sastras" (laws of planning kingdoms) that "a garden belt shall invariably surround a city, in the same way as a garden shall surround a house; the amenities and the actual

* Discussion on the paper by Stephen Child, M. Am. Soc. C. E., continued from September, 1928, *Proceedings*.

† Town Planning Asst., Madras, India.

‡ Received by the Secretary, July 18, 1928.

needs of a city will then benefit exactly as a house does from its garden." So one finds as a remarkable feature of all ancient Indian cities (towns and villages), the great boulevards and the dry garden belt circumscribing them, the beautiful avenues along roads and highways, and every house and building located in a garden site of its own. It is also enjoined in "sastras" that a site selected for city or town construction shall be planted with superior medicinal plants, trees, creepers, and vegetables, and shall in every way be a lovely place, a flat, even, and elevated country without burrow-pits and holes.

The history of India declares that by a careful selection of sites and planting of good trees in cities, people in the past enjoyed good health, long life, and intellectual and material prosperity. Thus, it seems essential that there must be some ideal to follow. It will be ludicrous, otherwise, to expect much "imagination or vision" in city planning. In the writer's opinion, the garden planning ideal of India should be also the future ideal of city planning in every other country in the world.

The home is, in truth, an important factor in a city plan. Next in importance are the essential public utility services. A city must rise or develop in the proper fashion strictly conforming to the various needs of a community, as to religion, society, politics, administration, commerce, industries, agriculture, and domestic life. Perhaps a city plan can never be complete, but provided the ideal is before the community, the city will be developing in the proper manner throughout the years.

City plans will fall under various systems, and the writer would classify them for simplicity into four principal systems:

1.—The limited—unitary system; that is, designing small cities of limited dimensions and providing for one center only. The ancient temple and commercial cities of India and presumably the cathedral cities of Europe are examples of such towns.

2.—The limited combination or joint system. In this case there may be two or more centers, but the city dimensions are limited. All the principal or capital cities of ancient India are of this system.

3.—The unlimited or free system, in which a city goes on developing without limit, innumerable centers being created as the need arises. Almost all modern cities, everywhere, are slowly developing to some extent on this system.

4.—The satellite system, in which several units—each planned independently of the other—are arranged about a principal or common center of activity. The original planning of kingdoms in India was of this system; that is, a small town was made the common center of ten villages or rural towns; a little larger town was made the common center of ten such small towns; a city was the common center of ten larger towns, etc. Sometimes, a city itself was planned so that the administrative block was at the center and the residential, commercial and educational blocks were arranged as satellites at some limited distances away from the center, with intervening park or garden belts.

Each of these systems in turn may adopt any of the following arrangements of streets: (*a*) The rectangular or square type; (*b*) the diagonal or radial type; (*c*) the circumferential or concentric type; (*d*) the segmental or

crescent type; and (*e*) the irregular type, or the "plan-as-you-go" type, which does not follow regularly any of the types mentioned. Each system and type of planning has a merit of its own and should suit particular needs of a community. Much study and standardization of systems and types for specific needs of a community are necessary, and this would mean the development of the fundamental science of city planning. India already possesses a comprehensive science of planning and, although it follows the ancient spiritual ideals of the nation, it has only to be applied and brought up to date to be in keeping with modern civilized ideals. The writer does not know whether America has any such science of planning for its guidance. Until the establishment of a comprehensive science of planning it must proceed with its developments very cautiously and city planning must certainly be very complex and extremely difficult in America.

It is the writer's belief that without the activities of the commission described by the author* a comprehensive civic survey would result in an inadequate city plan. From his experience in city and town planning work of various corporations and municipalities in Madras Province (South India), the writer has observed that slipshod methods that were in vogue in the past only resulted in abortive and unsuccessful plans. Now that all local authorities have been advised to prepare comprehensive civic surveys for their cities and towns, to formulate general development plans and regular and systematic programs of work to be undertaken in the due order of importance and urgency, he hopes conditions will be better. The preparation of the civic surveys is done under the guidance and supervision of the officers of the permanent Town Planning Department under the Government. A civic survey for one city (Vizagapatam) has been completed and published; civic surveys for eight more cities and towns are being prepared and many other cities and towns will also begin surveys sooner or later. A special staff, paid entirely by the local organizations, is working under the writer's direction, and this arrangement enables the preparation of the surveys in a very systematic and uniform manner and at great economy to the local bodies themselves.

There are also other objects for undertaking the preparation of civic surveys in advance of planning. Experience has shown that a fundamental necessity of human society is that workers should live reasonably close to their work. Before formulating a plan, it is necessary to ascertain where people work, where they live, and where congestion exists. In some cases it should be practicable to move the workshops. In others, sufficient accommodation may be found for the workers near their work by utilizing waste spaces or by raising buildings to several stories. Ordinarily, factories and indigenous trades which give subsistence to large numbers of working people should be located on the outskirts and not near the center of a city. A general neglect of the close correlation of the workshops and the residences of the workers has led to the failure of many planning programs otherwise well conceived.

Furthermore, the sharing of interests and responsibilities among the various communities and sections into which the population in India is divided, is proverbial, and it has been considered practicable, in carrying out a

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1012.

planning program, to deal with particular communities as corporate bodies rather than as individuals. By such means the detailed planning involved in allotting sites to, and collecting rentals from, a number of individuals can be facilitated, while at the same time the communities themselves, by their collective credit, are in a better position to raise funds for buildings.

The civic survey is now recognized as a necessary precedent to scientific and comprehensive planning of cities and towns. In future problems of zoning, etc., the survey would be found immensely useful, particularly if it is kept up to date. The responsibility for this initial survey as well as for its maintenance in an up-to-date form, has been placed upon the planning committees of the local bodies.

Under the present Town Planning Act of Madras Province, every Council has power to appoint special town planning committees to deal with special problems. Such committees may be composed wholly of members from the Council or outsiders may be co-opted up to one-third the total strength. It has been found desirable that planning in general should first be discussed and then specific plans of action decided, so that the question of constituting a general planning committee for each city and town with co-opted representatives of outside interests has been recommended in the case of several cities and towns.

The writer, therefore, fully agrees with the author that a city planning commission is an important adjunct to a city council. He would suggest further that all matters having any bearing on planning or improvement be referred to this commission and, if necessary, that it shall exercise the powers and perform the duties of the City Council, in so far as city planning is concerned, and that the presiding member thereof have the powers and duties of the Mayor. It would perhaps be of little use for an expert commission to adopt a general plan merely for development and leave the details of execution to the City Council. The preparation of a general plan is but a preliminary step and real difficulties arise when an attempt is made to achieve it. It would be well if the commission were both an advisory and executive body. The duty of enforcing the provisions of city planning legislation should also be vested in such a commission, but that may be subject to certain conditions and limitations. In such a case the commission may be constituted as follows: (*a*) Three members appointed by the Mayor; (*b*) three members elected by the City Council; (*c*) three representatives of other interests elected or nominated by transport companies, chamber of commerce, and similar bodies or associations; and (*d*) a president elected by the commission.

The writer, therefore, advises the creation of a city planning commission that would be in very truth "the permanent Board of Direction of the city", the decision of which on city planning matters shall be final.

M. W. Weir,* M. Am. Soc. C. E. (by letter).†—Every step in city planning has been and is the product of imagination; from the moment of the thought of a city the imagination responds in a succession of mental images. First

* Cons. Engr., New York, N. Y.

† Received by the Secretary, July 25, 1928.

is imaged a city built by plan, then follows imagined details of plan and methods of operation.

In planning and replanning the modern city, a great diversity of a citizen activity must perforce be given thought. There are, to-day, literally hundreds of individual activities in the business of living which must be considered in city planning if such planning is to serve its original purpose.

Of course these activities may be grouped under general headings and considered in imagination, treating such group to a general diagnosis of its needs; but such diagnosis is apt to produce unfairness to parts of each general group the activities of which partake of special requirements of some sort.

It may be said that to divide these general groups into such fine parts would tend to involve city planning in too much detail, but the final development of detail is the foundation of the building of the imagined whole. Of what use would be a beautifully designed machine if the bolts and screws that were meant to hold it together were loose or some of them had been omitted; it simply would not work.

The demands of city planning are so great that it is doubtful if ever one man lived who could imagine the problems of every phase of the plan. Discussion with other people and the use of other people's imagination is necessary to the city planner, to give his imagination the correct perspective of the varied occupations and needs of occupational and social activities. These occupational and social matters divide into many separate problems which influence the city plan in no uncertain way.

The problems of the city plan demand active imagination from specialists in many lines; for instance, the civil engineer must imagine and plan for streets, water supply and distribution, sewerage and sewage disposal, water-frontage development, transit and transportation, and bridges with all their details; the architect must imagine façades on streets, public buildings and their grouping, and the many architectural embellishments to the city; the landscape engineer must imagine the parks and playgrounds, the great boulevards, the gardens, the street tree arrangement, the vistas in the city, and many other details; the industrial engineer must imagine and plan for the expansion of industry and the most efficient application of land best adapted to industrial uses; the zoning engineer must imagine and plan for the uses of land and buildings; and so on almost indefinitely. All these imaginings and plans must be co-related and drawn into the city plan, a product of the imagination as are all things made by Man.

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## PAPERS AND DISCUSSIONS



# HYDRAULIC STUDIES AND OPERATING RESULTS ON THE MIAMI FLOOD CONTROL SYSTEM

## Discussion*

By B. F. Jakobsen, M. Am. Soc. C. E.

B. F. Jakobsen,† M. Am. Soc. C. E. (by letter).‡—The writer does not understand the statement§ that:

"The discharge coefficient, *c*, is slightly lower for the Germantown than for the Englewood conduits (Table 1). This fact is difficult to explain, as the interiors of the conduits at the two dams appear to be equally smooth." Since this coefficient depends both on the length and on the hydraulic radius of the two conduits, it is accidental that they are nearly alike.

The carefully made measurements afford an opportunity to check the formula for friction head, for water flowing with high velocity in a large conduit. On the assumption, made by the author, that the entrance loss is negligible, the total head is consumed, partly as velocity head and partly as friction, or,

$$h = \frac{v_a^2}{2g} + h_f \qquad (1)$$

in which,

$h$ = total head, in feet;
$v_a$ = actual velocity, in feet per second;
$2g = 64.36$; $\sqrt{2g} = 8.023$; and
$h_f$ = friction head, in feet.

The theoretical velocity head is $\frac{v_a^2}{2g}$ on the assumption that $v_a$ is constant for the entire section. The friction head is,

$$h_f = \frac{m}{2g} \frac{v_a^n}{(4r)^x} L \qquad (2)$$

* Discussion on the paper by C. H. Eiffert, M. Am. Soc. C. E., continued from August, 1928, *Proceedings*.

† Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

‡ Received by the Secretary, June 9, 1928.

§ *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1375.

in which, $L$ = length of conduit, in feet; $r$ = hydraulic radius, in feet; and $m$, $n$, and $x$ are constants. For new cast-iron pipe, Unwin gives,* $m = 0.0215$; $x = 1.168$; and $n = 1.95$. Assuming, however, that $n = 2$ (since that checks closely with the results found) and combining Equations (1) and (2):

$$h = \frac{v_a^2}{2\,g}\left(1 + \frac{m\,L}{(4\,r)^x}\right) \qquad (3)$$

Let,

$$k = \frac{m\,L}{(4\,r)^x} \qquad (4)$$

so that,

$$h = \frac{v_a^2}{2\,g}(1 + k) \qquad (5)$$

or,

$$v_a = c\,\sqrt{2\,g\,h} \qquad (6)$$

in which, $c$ is the author's constant in Table 1.† From Equations (5) and (6) it follows that,

$$c = \frac{1}{\sqrt{1 + k}}, \text{ or } k = \frac{1}{c^2} - 1 \qquad (7)$$

If Equation (6) is written,

$$c = \frac{v_a}{\sqrt{2\,g\,h}}$$

and it is assumed that every value of $c$ thus determined from corresponding measurements of $v_a$ and $h_1$ is equally probable, then the principle of least squares demands, that,

$$\sum\left(\frac{v_a}{\sqrt{2\,g\,h}} - c_m\right)^2 = \text{minimum} \qquad (8)$$

in which, $c_m$ is the most probable value of the constant, $c$. If $N$ is the number of tests, then,

$$c_m = \frac{1}{N}\sum\frac{v_a}{\sqrt{2\,g\,h}} = \frac{1}{N}\sum c \qquad (9)$$

or $c_m$ is the arithmetic mean of all values of $c$. It would probably be more correct to derive $c_m$ from Equation (5). This requires more work and the value thus found differs only by about 0.1% from the arithmetic mean used by the author. The probable error of $c_m$ is given by,

$$e = \frac{2}{3}\sqrt{\frac{\Sigma\,(c - c_m)^2}{N\,(N - 1)}} \qquad (10)$$

and the probable error of one measurement is,

$$e_1 = \frac{2}{3}\sqrt{\frac{\Sigma\,(c - c_m)^2}{(N - 1)}} \qquad (11)$$

The values of $L$ and $r$ for the two conduits are given in Table 1. From these are obtained for the Germantown conduit, $c_m = 0.79708$; $e = 0.0027$; and

* "A Treatise on Hydraulics," 1912, p. 217.

† *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1376.

$k = 0.5740$, whereas for the Englewood conduit, the values are $c_m = 0.8043$; $e = 0.0026$; and $k = 0.5458$.

For the Germantown conduit the probable error of the mean is $\frac{0.27}{0.797}$ $= 0.34\%$ and if the average velocity measured is about 7 ft. (since, presumably, the high velocities in the conduits were not actually measured), the error of the average is $7 \times 0.0034 = 0.0238$ ft. per sec. and the probable error of a single measurement is $\sqrt{12}$ times that, or 0.0825 ft. per sec. For the Englewood conduit about the same results are obtained. This indicates that the measurements were carefully made and that the friction head varies nearly as the square of the velocity.

The constant, $m$, can now be determined from Equation (4). That gives for the Germantown conduit,

$$m = 0.5740 \times \frac{14.24}{546} = 0.01497$$

and, for the Englewood conduit,

$$m = 0.5458 \times \frac{16.14}{712} = 0.01222$$

Both values are much smaller than the value given by Unwin for cast-iron pipe, and the value for the Germantown conduit is about 20% greater than that for the Englewood conduit. If this is to be accounted for by entrance loss, as the author suggests,* assuming that there is no such loss in connection with the Englewood conduit and that $m = 0.01222$ for the Germantown conduit, then,

$$h = \frac{v_a^2}{2\,g}\left(1 + 0.1222\,\frac{546}{14.24} + C\right)$$

Taking an average test, as, for example, $h = 18.55$ and $v_a = 27.2$, then, $1.614 = 1 + 0.468 + C$, or $C = 0.145$; that is, 14.5% of the velocity head is absorbed as entrance loss, or the entrance loss of the Germantown conduit is $\frac{0.145}{0.468}$, or 31.0% of the friction head, or the entrance head gives a loss equal to about 169 ft. of conduit.

The two conduits are built alike; the area of the Germantown conduit is 2 by 91 sq. ft. and that of the Englewood is 2 by 108.5 sq. ft. It seems unlikely that the constant, $m$, should vary 20% when determined from measurements on conduits that are so nearly similar. The entrance head of 14.5% seems also excessive, especially in view of the author's opinion that the entrance loss at both conduits should be very small. The writer hopes the author will be able to throw some light on this matter. He may possibly have tests from other reservoirs that may be of assistance.

As far as the writer knows, there is little information available concerning friction losses in large conduits for high velocities, and it seems, therefore, highly desirable that the meager data shall agree as closely as possible.

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1375.

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# MEMOIRS OF DECEASED MEMBERS

## GEORGE HAMPTON BREMNER, M. Am. Soc. C. E.*

Died April 3, 1927.

George Hampton Bremner was born at Marshalltown, Iowa, on December 16, 1861. His father, William Bremner, came to America from Eastern Scotland in early childhood with his parents who settled in New England. After preparing for both law and engineering, William Bremner was actively engaged for a number of years in railway construction, chiefly in Connecticut. Joining the westward movement, he migrated to the State of Iowa and there was married to Catherine Hampton, of Iowa City, who was of English stock and a direct descendant of Governor Chittenden of Vermont. In 1856, they settled in Marshalltown, Iowa, where they reared their family. William Bremner served in the State Legislature in the early Sixties and practiced his two professions for many years, serving as City Engineer and as County Surveyor at Marshalltown until the end of his active professional life.

As a boy, George Hampton Bremner became familiar with the technique of instruments and practical methods of construction by helping his father. Railroads presented an attraction for him, however, and he spent his vacations, while attending the University of Iowa at Iowa City, Iowa, as a Rodman with Middle Western railway companies. After his graduation from the University in 1883, he worked with his father for two years, and then entered the employ of the Chicago, Burlington and Quincy Railroad Company in 1885, as a Division Engineer at Red Oak, Iowa.

From 1889 to 1890 Mr. Bremner was in charge of a locating party on the St. Louis, Keokuk and Northwestern Railroad (a subsidiary of the Burlington), locating a line between Old Monroe and St. Louis, Mo., which section includes the crossing of the Missouri River near Bellefontaine, Mo. At the conclusion of this work he returned to the Chicago, Burlington and Quincy Railroad Company, and was placed in charge of the maintenance of lines in Northern Illinois with headquarters in Chicago. In 1898 the Burlington undertook its first track elevation (grade separation) at Chicago and Mr. Bremner was placed in charge. On this work he made an enviable record, taking special pride in the speed, economy, and quality in this pioneer construction work, while handling it with a minimum of interference with train operation. In 1904, he was transferred to the Operating Department as Engineer, Maintenance of Way, of the Illinois Lines. He returned to the Engineering Department in 1908 as District Engineer in charge of engineering work on the lines in Illinois, Wisconsin, and Minnesota.

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* Memoir prepared by a Joint Committee of the Society and the Western Society of Engineers, consisting of Elmer T. Howson, *Chairman*, Edwin F. Wendt, and W. D. Pence, Members, Am. Soc. C. E., and R. T. Scholes, M. W. S. E.

When the Bureau of Valuation was organized under the Interstate Commerce Commission in 1913, Mr. Bremner was appointed Assistant District Engineer in the Central District, with headquarters in Chicago. In 1915, he was promoted to District Engineer for this territory, and placed in charge of the administrative detail of field and office work, in connection with the preparation of inventories and reports of all the steam railway properties within this District. In the course of his service with the Interstate Commerce Commission, he was also called upon to make a number of special reports on railway properties for the Commission under resolutions of Congress. He terminated this service in December, 1921, when the District Offices were closed and the forces concentrated at Washington, D. C. After engaging for a short time in consulting practice, he returned to the Chicago, Burlington and Quincy Railroad Company, with which he remained until his death.

Mr. Bremner always took a keen interest in the work of Engineering Societies. He joined the Western Society of Engineers on October 8, 1887, and participated in the organization of the American Railway Engineering Association in 1897-99. He was a member of the Special Joint Committee of the Society and the American Railway Engineering Association on Stresses in Railway Track, from its organization, in 1913, until his death. In 1906, the Western Society of Engineers awarded him the Octave Chanute Medal for the preparation and presentation of a paper of special merit on the subject of "Areas of Waterways for Culverts and Bridges." Mr. Bremner took a particularly active part in the work of the American Railway Engineering Association, serving at various times as a member of the Roadway and Track Committees, as well as Chairman of the Roadway Committee for three years. He was elected Treasurer of the Association in 1911, in which capacity he served until his death.

Gifted with strong personal initiative, Mr. Bremner was nevertheless a good soldier in carrying out the wishes of a superior authority, when the decision and plan of action were made. It was his habit when approaching new problems to form his judgment carefully, at times even slowly; with this quality of standing firmly for his beliefs, he did not hesitate to recast his views in the light of new facts, of which he was ever in search on his own account. Of his many strong personal qualities none is better remembered than his sturdy sense of business integrity, a steadfast loyalty to his friends, and, above all, his fine devotion to his family circle.

As a boy, Mr. Bremner united with the Presbyterian Church, which organization received his unfailing loyalty throughout his life.

He was married on September 5, 1898, at Marshalltown, Iowa, to Louie A. Stephenson, who with two sons, Charles W. and George H., and a daughter, Mrs. Clarence Schaeffer, of Chicago, survives him; he is also survived by a sister, Mrs. O. A. Byington, of Iowa City, Iowa, and a brother, William H. Bremner, of Minneapolis, Minn.

Mr. Bremner was elected a Member of the American Society of Civil Engineers on December 6, 1899.

## ALVAH SEYMOUR GOING, M. Am. Soc. C. E.*

Died March 21, 1927.

Alvah Seymour Going was born at Portland, Ore., on April 7, 1860, the son of James Wallace and Isabella (Eads) Going. His father's family settled at Shirley, Mass., early in the Eighteenth Century, the family name then being Gowing. Mr. Going was educated in the public schools of his native city and for a time attended the Oregon State College.

In 1881 he began his railroad career as Levelman on location for the Oregon Railway and Navigation Company. From 1882 until 1883 he was employed as Transitman on construction of the Clark's Fork Division of the Northern Pacific Railway, and from March to October, 1883, he was in charge of exploration and location surveys for the Astoria and Forest Grove Railroad.

At this time the West was rapidly developing; many railroad projects were under way and this character of work fascinated Mr. Going. He was engaged on many of these projects in Oregon, Washington, Idaho, and in British Columbia, Canada, from 1881 to 1890, when he formed a partnership with Mr. A. W. Miller and entered the general practice of engineering at Port Townsend, Wash. During his association with Mr. Miller he served as Engineer of several small railroads then under construction and also for the Victoria, Port Crescent and Chehalis Railroad, a line about 150 miles in length.

In 1891 Mr. Going moved to Victoria, B. C., Canada, where he engaged in general engineering practice but specialized in the exploration and location of railroads. This practice included work on many small mining railroads and for a time he was engaged on collieries on Vancouver Island.

In 1903 Mr. Going was employed as Reconaissance Engineer for the projected Grand Trunk Pacific Railway, to explore various routes through the Rocky Mountains, in advance of location survey parties. On these expeditions he was obliged to travel alone, except for Indian guides whom he employed from time to time. This exploration work was started late in 1903, making it necessary for him to spend most of a severe winter alone in a hostile, uninhabited country. He started this expedition with two horses, but was obliged to abandon them and complete the work on foot, arriving at Fort George, B. C., Canada, after many hardships, but in possession of much invaluable information regarding the passes through the Continental Divide. Few engineers could have endured the hardships of this work. It was in this class of occupation that he found the greatest interest. He was at his best when confronted with the problem of finding a location for a railroad through new territory.

In October, 1905, Mr. Going was engaged by the Minneapolis and St. Louis Railroad Company in South Dakota, where he located and built 130 miles of line from Conde to LeBeau on the Missouri River.

During 1907 he became associated with the Grand Trunk Railway Company of Canada as Locating Engineer, with headquarters at Montreal, Que.

* Memoir prepared by R. D. Garner, Esq., Providence, R. I.

As such, he supervised all the location work under way at that time, notably the 85-mile extension of the Central Vermont Railway to Providence, R. I. When the Grand Trunk Railway and the Canadian Northern Railway were merged into the Canadian National Railways, Mr. Going was retained in the Bureau of Economics as Terminal Engineer. He held this position until his death.

Mr. Going is best remembered by those who were fortunate enough to be associated with him for his unfailing energy in helping younger engineers. He was never too busy or too tired to lay aside the work in hand to aid and counsel others. He possessed a remarkable memory which was combined with an almost uncanny ability to know exactly where to look for the engineering information that might be wanted. He was an omnivorous reader. No technical publication escaped him and from these he made innumerable notes and references for future use. In disposition, he was the personification of modesty, and ever tolerant of the defects in the characters of others.

Mr. Going was a member of the Masonic fraternity. He was also a member of the American Railway Engineering Association and served on its Committee on the Economics of Location for many years, having devoted a great deal of study to this work.

While in the South, in December, 1927, Mr. Going was taken seriously ill. He returned to Montreal, where after a lingering illness he died at the Royal Victoria Hospital on March 21, 1927. He is survived by his widow, Harriett Jackson Going, and one daughter, Mrs. Chase Going Woodhouse, of Washington, D. C.

Mr. Going was elected an Associate Member of the American Society of Civil Engineers on May 4, 1892, and a Member on June 7, 1899.

## JAMES ORMEROD HEYWORTH, M. Am. Soc. C. E.*

DIED MARCH 15, 1928.

James Ormerod Heyworth was born in Chicago, Ill., on June 12, 1866, the son of James O. and Julia F. (Dimon) Heyworth. He was educated in the public schools of Chicago and was graduated from Yale University, New Haven, Conn., in 1888.

Mr. Heyworth was both an Engineer and a Contractor, a great amount of the work carried out by him as Contractor having been designed under his personal direction as well. He became interested in concrete construction early in has career and was the Contractor on some of the earlier undertakings in which reinforced concrete was used in the United States. Among other work, he designed and built the first concrete building in Chicago, in 1904.

He was always interested in efficient construction and accordingly underwent a rigorous training in the field, spending considerable time on the work, in order to keep in close personal contact with it. He was a member of the firm of Christie, Lowe, and Heyworth from 1897 to 1903, after which

* Memoir prepared by the following Committee of the Illinois Section: I. F. Stern, *Chairman*, A. J. Hammond, and O. E. Strehlow, Members, Am. Soc. C. E.

he operated under his own name. During the last few years his activities were carried on under the name of James O. Heyworth, Inc.

Mr. Heyworth was particularly interested in difficult and hazardous engineering undertakings, or in those in which speed of construction was one of the essential factors. Heavy excavation and difficult foundation and concrete work, railroad track elevation, bridges, hydro-electric plants, locks, dams, piers, wharves, docks, canals, tunnels, and caissons, both in the United States and in Canada, constituted the bulk of his work.

In the course of his career, Mr. Heyworth was called upon to restore and rebuild eight dams which had failed. In every case the work was carried out safely and successfully under his direction.

His desire to be engaged in difficult enterprises was displayed in the past few years in the building of the concrete arch bridge over the Mississippi River between St. Paul and Minneapolis, Minn. In North Carolina, he achieved a record, not only for speed in concrete road mileage built during one season, but also for the uniform excellence of the work.

Mr. Heyworth had many inventions to his credit, among others being that of the modern dragline excavator, which was developed by him.

He was a great believer in team work and during his years of activity was assisted by a loyal organization working under his direction. He was an indefatigable worker, personally laying out details of plant and equipment on his contract work and always eager to apply new methods of merit in its prosecution. He was one of the first to use a central flexible electric power plant on large construction work and was a great believer in the use of Diesel engines for this purpose.

Mr. Heyworth's activities, however, extended beyond engineering and contracting lines. He was deeply interested in civic and philanthropic affairs, full of human sympathy, an enthusiastic yachtsman and fisherman, and a lover of the great outdoors. Some years ago he was President of the Local Branch of the Izaac Walton League of America and also Commodore of the Chicago Yacht Club.

He was instrumental in securing the present quarters of the Chicago Engineers Club. When the opportunity offered itself, he took an option on the property in his own name, without consulting the members of the Board, and called a meeting of the prominent members of the Club. By the time the Club membership voted to secure quarters of its own, the value of this property had risen very considerably. Mr. Heyworth then turned his option over to the Club and, as a result of the increased value of the property, there was no difficulty in floating a bond issue. During these trying times, Mr. Heyworth was induced to take the Presidency, which he held for three consecutive years at a great personal sacrifice to himself. To him, more than to any other man, credit should be given for enabling the Engineers Club to acquire its own quarters.

Mr. Heyworth was always enthusiastic in regard to every enterprise with which he was connected. He was intensely loyal to his Alma Mater, Yale University, from the day of his graduation to that of his death and always a "prime mover" in anything pertaining to her interests. He was a member of

the Executive Committee of the Yale Advisory Board; Chairman of the North Central Division of the Yale Endowment Fund; and a member of the Yale Clubs of Chicago and New York, as well as of other college organizations and clubs.

In 1917, he heeded the call to assist in war activities and accepted the position of Manager of the Division of Wood Ship Construction of the Emergency Fleet Corporation, United States Shipping Board, which he held until after the end of the World War. During this period he turned his own contracting business over to his subordinates and devoted himself entirely to the work of his Department of the Shipping Board, which, under his direction, showed a great record for efficiency. For some time, before the end of the war, fifty ships completely equipped, ready for service, were being turned out each month.

In 1902, Mr. Heyworth was married to Martica Gookin Waterman. They had two children, Francis Dimon and James O., Jr.

He was in the prime of his powers at the time of his death. He was taken ill in August, 1927, and was ordered by his doctors to Arizona with the hope that the dry climate would restore him to health. For a time he made progress toward recovery, but later complications set in and he grew steadily worse. He asked to be taken home to Chicago where he passed away a few hours after his arrival.

His death is deeply mourned, not only by his family and friends, but by a host of younger engineers and contractors who came to him for advice and help in their troubles and to whom he gave an unstinting measure of his broad training and ripe experience.

Mr. Heyworth was elected a Member of the American Society of Civil Engineers on May 6, 1914.

## RUDOLF VIEDT ROSE, M. Am. Soc. C. E.*

Died January 25, 1928.

Rudolf Viedt Rose, the only son of Adolf and Helene (Viedt) Rose, was born in Niagara Falls, N. Y., on April 27, 1876. He was of fine old German stock, both his parents having come from Germany. His father left his native town of Hameln, in the Province of Hanover, to come to America, and settled at Niagara Falls where he became an honored and successful business man.

Mr. Rose was educated in the public schools of Niagara Falls and the Central High School, of Buffalo, N. Y. His engineering education was obtained in Stevens Institute of Technology, at Hoboken, N. J., from which he was graduated in 1897 with the degree of Mechanical Engineer.

Immediately after graduation, Mr. Rose began his active work as Draftsman with a manufacturing company in Buffalo, but shortly thereafter on September 15, 1897, he returned to Niagara Falls to enter the Engineering

* Memoir prepared by Walter McCulloh, M. Am. Soc. C. E.

Corps of the Niagara Falls Power Company. He remained with this Company for twenty-three years (until and following its consolidation with the Hydraulic Power Company on June 1, 1920), filling many positions from Draftsman to Construction Engineer in connection with the construction and operation of the now famous hydro-electric power development on the Niagara River. During his earlier employment with the Power Company, he spent a six months' leave in Germany, working with Allgemeine Electricitäts Gesellschaft at Berlin.

In 1920, Mr. Rose entered commercial life as President and Manager of the Consolidated Fuel and Supply Corporation, which he conducted for several years, dealing in coal, contractors' supplies, and building materials. His company was then merged with the Empire Builders' Supply Company in Niagara Falls, and he held the position of Engineer and Superintendent in this larger industry until his death.

He was intensely interested in his professional work and always active and energetic. His was a pleasing personality and he endeared himself to a large circle of friends.

In 1907, Mr. Rose was married to Joyce Grant, of Toronto, Ont., Canada, who, with their three children, Carl Grant, Rudolf William, and Joyce Helene, survives him.

He was a member of the Niagara Club, the Niagara Falls Country Club, and the Honorary Society of Tau Beta Pi. He was a Vestryman of St. Peter's Protestant Episcopal Church.

Mr. Rose was elected an Associate Member of the American Society of Civil Engineers on April 5, 1905, and a Member on February 28, 1911.

## PAUL BERTRAM TALLMAN, M. Am. Soc. C. E.*

Died July 29, 1926.

Paul Bertram Tallman was born at East Orange, N. J., on January 24, 1884. He was the son of the late Stephen Sands Tallman and Martha Broadmeadow Tallman. His great-grandfather, Simeon Broadmeadow, came to the United States from England in 1825 and by a Special Act of Congress he was made a citizen. He was one of the early promoters of the manufacture of steel and gas in this country, besides being an inventor of National fame.

Mr. Tallman's early life was spent in East Orange, where he fitted himself for college. On account of ill-health, he was forced to abandon his course, but from this time forward, he became a student and completed his education as an engineer even more thoroughly than college curricula require. His daily work in engineering was followed with careful study of the related problems during his hours of leisure. His greatest recreation was the complete and successful study and solution of these problems and the systematic recording of his findings.

* Memoir prepared by P. M. Corry, M. Am. Soc. C. E.

His practical engineering experience began during the summer of 1902, with the Clearfield Bituminous Coal Company at Rossiter, Pa. In January, 1903, he entered the service of the New York Central and Hudson River Railroad Company, in the Terminal Engineer's Office, where he served, successively, as Chainman, Rodman, Instrumentman, and Draftsman on work in connection with the Grand Central Terminal improvements. In 1906, he resigned his position with the Railroad Company and accepted that of Engineer and General Superintendent with Bunn and Nase, General Contractors, of New York, N. Y.

From 1909, until his death, Mr. Tallman was in the employ of the firm of Warren and Wetmore, Architects, of New York City. While in this position, his work was of a most varied character and comprised largely the design and installation of mechanical equipment and plants, and the supervision of the construction of numerous buildings, principal among which were the Union Station, Houston, Tex.; the Broadmoor Hotel, Colorado Springs, Colo.; the Ritz-Carlton Hotel, Atlantic City, N. J.; and the Plaza Addition, the Heckscher Building, Aeolian Hall, and the Goelet Building, New York City.

In 1923, when a new department was organized by the firm for conducting architectural and engineering supervision for loaning institutions, Mr. Tallman was selected as its head. Here, again, his records and studies were of great assistance in organizing and conducting the work. Notable among the buildings under supervision by this Department, were the Cooper Department Store Building, Los Angeles, Calif.; the Michigan Theatre and Office Building, Detroit, Mich.; the Penn Athletic Club, Philadelphia, Pa.; and 200 Madison Avenue, and the new Paramount Theatre and Office Building, New York City.

The high esteem in which Mr. Tallman was held by his fellow members of the Building Committee for the new Paramount Theatre and Office Building, is shown in the following resolutions:

"*Whereas,* on the 29th day of July, 1926, there departed this life in Rockville Center, Long Island,

PAUL B. TALLMAN

one of the Members of the Building Committee of The Paramount Building, New York City. At a meeting of the said Committee duly held on the 2nd day of September, 1926, it was

"*Resolved,* that out of respect to the memory of the deceased, suitable action should be taken to mark the high regard in which he was held at all times as a gentleman and engineer of ability and wise counsel; and

"*Resolved,* that we deeply deplore his death and extend to his widow and son our deepest sympathy in the hour of their great grief; and

"*Resolved,* that a copy of these resolutions be forwarded to his family."

On October 21, 1909, Mr. Tallman was married to Edna Davey, of East Orange, who, with a son, Stephen, survives him.

Mr. Tallman was of a highly sociable nature and enjoyed the company of his many friends. He was an untiring worker and never spared himself when his time and energies were needed by his friends or business associates.

In business dealings he never took advantage of another's mistakes. He was instinctively honest and fair. His oldest friends were his best friends.

He took an active interest in civic and religious affairs. He was a life-long member of the Presbyterian Church—the last four years a devoted member of the Rockville Centre Presbyterian Church. He was President of the Board of Trustees and also very active in the Men's Club and the Sunday School.

In 1922, he was elected a Fellow in Perpetuity by the Metropolitan Museum of Art, New York City. He was also a member of the American Railway Association.

Mr. Tallman was elected a Junior of the American Society of Civil Engineers on October 3, 1905, an Associate Member on September 10, 1910, and a Member on May 28, 1923.

## GUY BANKER EDWARDS, Assoc. M. Am. Soc. C. E.*

Died November 4, 1927.

Guy Banker Edwards, the son of Dr. John Edwards and Myra (Banker) Edwards, was born at Gloversville, N. Y., on April 6, 1875. His boyhood years were passed in Gloversville, where he received his education in the public schools. He then attended the Clinton Liberal Institute at Fort Plain, N. Y. After his graduation from this preparatory school, he attended Philips Academy at Andover, Mass., and, later, continued his studies in Civil Engineering at Union College, Schenectady, N. Y., from which institution he was graduated in 1895.

From 1895 to 1906 Mr. Edwards held the position of Inspector of Public Works or Assistant Engineer at Gloversville. From 1906 to the spring of 1909, he was employed as Construction Engineer and Superintendent with Townsend and Fleming, Landscape Architects, of Buffalo, N. Y. During this time he was in charge of important and extensive landscape construction programs on a number of large private estates.

In 1909 he entered the employ of Mr. Warren H. Manning, Landscape Architect, in Boston, Mass. During his association with Mr. Manning he was in charge of large private estate construction work. In October of that year and during 1910, he was employed in the operation of stone quarry and stone sawing plant.

From April to December, 1911, Mr. Edwards was Resident Engineer of the Iowa Engineering Company at Clinton, Iowa, in which position he was in charge of asphaltic concrete paving and sewer system and disposal plant construction. He later accepted a most important position with F. A. Barbour, M. Am. Soc. C. E., of Boston, and E. G. Bradbury, M. Am. Soc. C. E., of Columbus, Ohio, as Resident Engineer in charge of extensive water supply developments, including a large distributing reservoir for the City of Akron, Ohio.

* Memoir prepared by Albert D. Taylor, Landscape Archt., Cleveland, Ohio.

From July, 1915, to June, 1916, Mr. Edwards was Supervising Engineer on sewage and garbage disposal plant construction at Akron, for the late R. Winthrop Pratt, M. Am. Soc. C. E., Construction Engineer, of Cleveland, Ohio. Immediately following this engagement he accepted a position as Resident Engineer for the Cleveland and Youngstown Railroad Company in charge of important construction work, and from September, 1921, to September, 1923, he was employed by Mr. A. D. Taylor, of Cleveland, in charge of general construction work and as Resident Engineer of several housing developments in the Gogebic Range of Northern Michigan.

From September 1, 1923, to the time of his death, Mr. Edwards was in the employ of the Oglebay-Norton Company as Efficiency Engineer. The work for which he was responsible during the last two or three years was as follows: Insurance, safety, welfare and housing problems for iron ore mines under the Oglebay Norton and Company management; Montreal, Ottawa, Eureka-Asteroid, Berkshire, Bristol, and Feigh mines.

During his work as a Civil Engineer, Mr. Edwards exhibited sterling qualities of mature manhood and profound knowledge of his field of activity. No one in the Engineering Profession who came in contact with him had other than the highest respect for his engineering ability, his seriousness of purpose, his principles of honesty, and the consistent application of his physical and mental energy to the completion of the tasks for which he was made responsible.

Mr. Edwards was elected an Associate Member of the American Society of Civil Engineers on March 13, 1917.

## VIRGIL SAMMS ONSTOTT, Assoc. M. Am. Soc. C. E.*

Died November 11, 1927.

Virgil Samms Onstott, the son of Dr. Elmer Onstott and Emma J. Onstott, was born on September 22, 1897, at Saltsburg, Pa. He was graduated from Saltsburg High School in June, 1915, and in the fall of that year he entered Kiskiminetas Springs School from which he was graduated with high honors in 1917, winning a medal for proficiency in mathematics as well as a scholarship conferred by the Cornell Alumni Association of Western Pennsylvania.

In September, 1917, Mr. Onstott entered Cornell University at Ithaca, N. Y. He enlisted in the Students' Army Training Corps, on October 8, 1918, and was assigned to Company H of the Reserve Officers Training Corps, from which he received his discharge the following year. In 1921, he was graduated from Cornell University with honors and a degree in Civil Engineering.

Subsequent to his graduation he was employed by the Pennsylvania Department of Highways first as Inspector and, later, as Senior Inspector in charge of various kinds of pavement construction work. In 1924 he was

* Memoir prepared by J. L. Herber, M. Am. Soc. C. E.

engaged as Assistant Resident Engineer in charge of State Highway construction in Beaver County, Pennsylvania, with headquarters at Pittsburgh and at Rochester, Pa., which position he held until his death.

Mr. Onstott was a member of the Presbyterian Church of Saltsburg with which he was affiliated from his early youth. On December 26, 1923, he was commissioned as Second Lieutenant in the United States Engineer Officers Reserve Corps.

He is survived by his widow, Margaret Anne Onstott, and one son aged 19 months, also his parents and one brother, Howard K. Onstott, of Cleveland, Ohio, and one sister, Mrs. G. G. Waite, of Toronto, Ont., Canada.

Mr. Onstott not only had all the qualifications of a good engineer, but a character which presaged his success. His loyalty and his steady purpose of co-operating in any undertaking which was assigned to him, commanded the respect and admiration of his associates. His calm judgment and unpretentious manner were noticeable in everything he undertook so willingly. It was, therefore, only natural for him to analyze his problems thoroughly—a characteristic so essential in a good engineer. His passing is a distinct loss to the profession.

Mr. Onstott was elected an Associate Member of the American Society of Civil Engineers on March 14, 1927.

## JOHN GANSOVERTTE ROSE, Assoc. M. Am. Soc. C. E.*

Died October 27, 1927.

John Gansovertte Rose was born on April 11, 1878, in Hutchinson, Kans. He was graduated from the Hutchinson High School, and, later, from Nickerson College, where he completed a post-graduate course in 1903, and subsequently taught for three years. After holding various engineering positions until 1907, he entered the University of Colorado at Denver, Colo., from which he received the degree of Bachelor of Science in Civil Engineering in 1911.

During his vacation periods in 1909 and 1910, Mr. Rose assisted in both field and office for the firm of Field, Fellows, and Hinderlider, of Denver, Engineers for the Orchard Construction Company. This project includes 12 000 acres of land near Grand Junction, Colo. The irrigation system embraces a movable diversion dam 380 ft. long, extending across the Grand River; 9 miles of power canal, about 6 miles of which were bench and trestle flume; a hydraulic power plant consisting of four units of Leffel turbines, directly connected with centrifugal pumps, two of which operated a lift of 75 ft. and the other two, a lift of 125 ft.; 40 miles of distributing canals; 1 800 ft. of concrete-lined tunnel; and various other structures, such as inverted siphons, concrete culverts, highway bridges, waste-gates, etc. The estimated cost of this varied construction work was $1 250 000.

* Memoir prepared by W. L. Prouty, Assoc. M. Am. Soc. C. E., and A. E. Palen, M. Am. Soc. C. E.

In June and July, 1911, Mr. Rose served as Engineer for the Badita Reservoir Company for which he made stream measurements and hydraulic investigations. He also submitted reports on a reservoir site in Southern Colorado.

From September, 1911, to May, 1912, he was employed by the Goldsborough Company, Engineers for the Laramie Water Company of Laramie, Wyo., on a 150 000-acre project. During this period he acted as Instrumentman on the preliminary and final location of canals having capacities of 1 000 sec-ft. and less. In June, 1912, he became Assistant Engineer in charge of the construction of canals for the Goldsborough Company. One of these, a 350 sec-ft. canal, 9 miles long, was located in a rough mountainous country covered with a dense growth of pine timber, the final estimated cost of construction of which was $168 000.

In November, when work on the canal had to be abandoned on account of the weather, Mr. Rose entered the employ of the Kansas City Structural Steel Company, of Kansas City, Mo. During the remainder of this year, and until May, 1913, he acted as Structural Detailer on smelter and railroad buildings, including stairways and hip-and-valley work.

From June, 1913, to October, 1915, he served as Engineer and Draftsman with the Consolidation Coal Company of Jenkins, Ky., making title maps of 100 000 acres of coal lands in the Cumberland Mountains. He also supervised the compilation of a complete index system of all notebooks (approximately 2 000), maps, and records of these lands, which had been collected throughout a period of forty years.

From November, 1915, to March, 1916, Mr. Rose served as Draftsman for the Kennicott Company of Chicago, Ill. This Company was engaged in the manufacture of water softeners, rapid filters, boilers, gas retorts, etc., which involved considerable pipe-fitting work, straight and spiral stairways, and varied similar construction, on all of which he had assignments.

In March, 1916, he became Draftsman for the American Bridge Company at Gary, Ind. During his engagement with this firm, his work included the preparation of shop drawings for highway bridges and for a large blast furnace and rolling mill.

In September, 1916, he was appointed Assistant Engineer with the Atchison, Topeka, and Santa Fé Railway Company, continuing until June, 1918, on the location survey of a double-track line, 65 miles long, between Carrollton and Moberly, Mo., to join the surveys of the Burlington Road for a new joint railway between Kansas City and St. Louis, Mo. This location was controlled by a definite list of limitations, such as (1) distance evaluated at $200 000 per mile; (2) curvature evaluated at $480 per degree; (3) rise and fall evaluated at $700 per ft.; (4) maximum grade, eastbound track, 0.3%; and (5) maximum grade, westbound track, 0.4 per cent.

As may be inferred, this project involved some heavy construction work (single cuts amounting to 500 000 cu. yd.), such as an under-crossing of the Wabash Railroad Terminal Yards at Moberly, and also included a large reservoir for water supply. Messrs. V. C. Coffey, Location Engineer, and Woodbury Howe, Chief Location Engineer, were in charge of this unusually complicated undertaking.

During June and July, 1917, Mr. Rose located a line, about 70 miles long, on the plains of Western Kansas, between Satanta and the Colorado State Line. From August to November, he made a close preliminary survey of a line between Kingfisher, Okla., and Shattuck on the Texas border (a distance of about 135 miles). From November until the early spring of 1918, he located a line (14 miles long) between Cassoday and Bazarr, in Eastern Kansas, and a belt line, with two alternate locations, around the Town of El Dorado, Kans., in order to relieve the traffic from the adjoining oil fields. Messrs. J. C. Beye acted as Location Engineer and H. C. Steward, as Chief Location Engineer, on the Kansas and Oklahoma surveys.

This continued engagement with the Santa Fé Railway Company gave Mr. Rose a broad and valued range of experience in railroad location. His duties during the two years of service covered all phases of the work required in making surveys, maps, profiles, estimates, and reports.

From February to June, 1918, he was associated with the United States Reclamation Service on the Rio Grande Project, at El Paso, Tex., in responsible charge of canals, farm laterals, and topographic surveys. He also made estimates of the cost of reconstruction and co-ordination of 40 miles of some old Spanish canals in the El Paso Valley.

During the summer he served as Assistant Engineer with the Idaho Irrigation Company of Shoshone, Idaho, in charge of a system of distribution canals and the replacement and installation of concrete head-gates, checks, drops, etc. During September and November, he was engaged with Lyman E. Bishop, M. Am. Soc. C. E., of Denver, Colo., as Resident Engineer in charge of construction of canal structures for the Meadow Farms Company, at Hardin, Colo.

In March, 1919, Mr. Rose was engaged as Chief of a Road Survey Party with the United States Bureau of Public Roads, District No. 3, with headquarters at Denver, in charge of the Deadwood-Hotsprings Project in the Black Hills in Lawrence, Pennington, and Custer Counties, South Dakota. This project was about 70 miles in length. Location surveys were made over about 60 miles, and construction was then in progress on approximately 16 miles of the road. His duties embraced the location and design of the road and all its structures, as well as supervision of construction, part of which is now being done under contract and part by force account.

In addition to this work, Mr. Rose was called on to consult and advise with local and county officials, as well as State and Forest Service officials, in various road matters relative to the Federal Aid Road Act. In this connection he had an opportunity to inspect numerous types of road construction and materials, to advise in matters of administrative organization and methods of management, and to inspect and check plans for other projects. As a matter of special interest it was his privilege to conceive and develop a new method of mass diagram which also serves as a haul sheet and furnishes an excellent means of balancing cut and fill quantities. This graph has been used extensively throughout District No. 3, and has received much favorable comment.

In 1921, Mr. Rose was appointed Assistant Highway Engineer and was sent to Washington, D. C., to receive special training as Materials Engineer

on Federal Aid Construction in the States of Wyoming, Colorado, and New Mexico. He held this position until his death.

Mr. Rose was married on September 15, 1912, in Denver, to Marie K. Venemann who, with four children, Jennie May, Helen Marie, Charles R., and Richard W., survives him. He is also survived by two sisters and two brothers.

Mr. Rose was a member of Union Lodge No. 7, A. F. and A. M., of Denver. He was also a member of the Colorado Section of the Society, and his death was felt as a personal loss to every member of the Section and to all who were in any way brought into contact with him.

A man who was loved by all who knew him, he was identified by his kindly ways as a genuine friend to his many acquaintances and business associates. In his home, he was a true husband and a real father; in the business world, he was a capable and efficient engineer.

Mr. Rose was elected an Associate Member of the American Society of Civil Engineers on April 14, 1919.

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## FRANK ANDREW TILLMAN, Assoc. M. Am. Soc. C. E.*

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Died September 2, 1927.

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Frank Andrew Tillman, the son of Peter M. and Ida Tillman, was born on September 27, 1888, at Waverly, N. Y. His early education was obtained in the Waverly Public Schools, after which he attended the University of Colorado at Denver, Colo., for two terms from October, 1908, to June, 1909, and from September, 1909, to June, 1910, taking special night and part-day studies in mathematics, plain surveying, municipal and irrigation engineering, construction, and design.

During the summers of 1908 and 1909, Mr. Tillman was engaged as Chainman, Rodman, and Topographer on preliminary and location surveys for the Denver, Laramie, and Northwestern Railroad, and as Rodman on Construction.

In June, 1910, he accepted a position as Transitman in charge of party on a land survey of 65 000 acres, comprising section line, irrigation ditch, fence line, building location, and general development, near Laramie, Wyo. He also spent some months on coal mine surveys at Medicine Bow, Wyo., and on the general layout of the Town of Milliken, Colo. In the fall of 1911 he acted as Levelman, for the Goldsborough Engineering Company, of Chicago, Ill., on a topographical survey of 30 000 acres in Wyoming, later, becoming Draftsman and Computer for this Company in Denver, Colo., on irrigation structures, lateral and canal projects, etc.

From January to April, 1913, Mr. Tillman was employed as Office Engineer for the Denver Realty Company on sub-divisions and utilities. He then returned to the Goldsborough Engineering Company as Chief of Construction on irrigation structures, canals, and laterals, until September, 1913, when he

* Memoir prepared by John A. Giles, M. Am. Soc. C. E.

was engaged as Engineer for the Enalpac Oil and Gas Company on the design, location, and construction of pump-houses, refineries, derricks, and pipe lines, at Casper and Cody, Wyo.

In July, 1915, he came East to Binghamton, N. Y., where he was employed from July, 1915, to May, 1917, by the City as Instrumentman on preliminary surveys and location for an intercepting sewer and other municipal work. He subsequently became Assistant City Engineer for the City of Binghamton in charge of sewer construction and design.

In September, 1918, Mr. Tillman entered the United States Government Service at Perth Amboy, N. J., on housing projects, topography, utilities, and street planning. The following year, however, he returned to Binghamton, where he acted as Secretary to the Commissioner of Public Works and Superintendent of Parks.

In 1920, he accepted a position as Assistant Engineer with Hoadley and Giles, a civil engineering firm of Binghamton, in charge of field parties and on general engineering work. He assisted in the design of the water supply and sewer systems of Johnson City, N. Y., and designed the proposed water system for Endicott and West Endicott, N. Y. He also assisted in the surveying and planning of the various subdivisions, and served as Office Engineer on computations and design.

In April, 1921, Mr. Tillman was appointed Commissioner of Public Works and Engineer of Johnson City, in charge of the design, construction, and maintenance of all public utilities. He held this position until his sudden death on September 2, 1927, from intestinal pneumonia.

In 1921, he was married to Lois A. Kilmer, of Binghamton, who, with a daughter, survives him.

Mr. Tillman was a member of the Baptist Church and prominent in the Masonic Fraternity as a member of Waverly Lodge, A. F. and A. M., the Scottish Rite Bodies, of Cheyanne Wyo., and Kalurah Temple, Nobles of the Mystic Shrine, of Binghamton, N. Y. He was also a member of the Kiwanis Club of Binghamton, N. Y.

He was most popular among his many friends and associates in the Engineering Profession, all of whom liked him for his genial disposition and respected him for his ability and thoroughness as an engineer. As a public official, he incurred the good-will of his associates and commanded the respect of the public.

Mr. Tillman was elected an Associate Member of the American Society of Civil Engineers on May 28, 1923.